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407-444

CONNECTICUT RIVER BASIN

FARMINGTON, CONNECTICUT

FARMINGTON RESERVOIR DAM
CT 00263

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

SEPTEMBER, 1980

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

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19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY,

Connecticut River Basin
Farmington, Connecticut

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The Farmington Reservoir Dam is an earth embankment impounding an unnamed tributary to the Pequabuck River and is reported to have been built in 1895. The dam is 760 feet long and 8 ft. wide at the top. In accordance with the Army Corps of Engineers Guidelines, Farmington Reservoir Dam is classified as a high hazard, small size dam. The test flood range is from one-half the PMF. Based upon the visual inspection at the site and past performance, the dam is judged to be in poor condition.

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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02254

REPLY TO
ATTENTION OF:
NEDED

DEC 19 1980

Honorable Ella T. Grasso
Governor of the State of Connecticut
State Capitol
Hartford, Connecticut 06115

Dear Governor Grasso:

Inclosed is a copy of the Farmington Reservoir Dam (CT-00263) Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Department of Environmental Protection, the cooperating agency for the State of Connecticut. In addition, a copy of the report has also been furnished the owner, The Farmington Water Company, Farmington, Conn. 06032.

Copies of this report will be made available to the public, upon request, by this office under the Freedom of Information Act. In the case of this report the release date will be thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Department of Environmental Protection for your cooperation in carrying out this program.

Sincerely,

WILLIAM E. HODGSON, JR.
Colonel, Corps of Engineers
Acting Division Engineer

Incl
As stated

CONNECTICUT RIVER BASIN

FARMINGTON, CONNECTICUT

FARMINGTON RESERVOIR DAM
CT 00263

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

SEPTEMBER, 1980

BRIEF ASSESSMENT

PHASE I INSPECTION REPORT

NATIONAL PROGRAM OF INSPECTION OF DAMS

Name of Dam:	FARMINGTON RESERVOIR DAM
Inventory Number:	CT 00275
State Located:	CONNECTICUT
County Located:	HARTFORD
Town Located:	FARMINGTON
Stream:	UNNAMED TRIBUTARY TO PEQUABUCK RIVER
Owner:	FARMINGTON WATER CO.
Date of Inspection:	MAY 12, 1980
Inspection Team:	PETER HEYNEN, P.E.
	MIRON PETROVSKY
	JAY A. COSTELLO
	MURALI ATLURU, P.E.
	JEFFREY BORNE

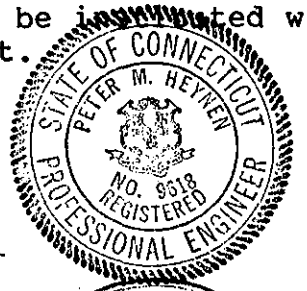
The Farmington Reservoir Dam is an earth embankment impounding an unnamed tributary to the Pequabuck River and is reported to have been built in 1895. The dam (including spillway) is 760 feet long and eight (8) feet wide at the top. The dam has a maximum impoundment capacity of 64 acre-feet and is about 8 feet in height from the toe of the downstream slope to the top of the dam. The spillway is a 32 foot long, and 2.8 foot high, concrete, ogee shaped weir. There is no information available on outlet pipes.

In accordance with the Army Corps of Engineers Guidelines, Farmington Reservoir Dam is classified as a high hazard, small size dam. The test flood range is from one-half the Probable Maximum Flood (½ PMF) to the Probable Maximum Flood (PMF). Based upon the potential downstream hazard, the test flood is selected to be equivalent to the ½ PMF. Peak inflow to the impoundment at the test flood is 315 cfs; peak outflow is 250 cfs with the maximum stage in the reservoir at 383.7, or 1.0 foot below the top of the dam. Based on this information the dam is not expected to overtop at the test flood condition. The spillway capacity with the pool at top of dam is estimated to be 525 cfs which is greater than 100% of the routed test flood outflow.

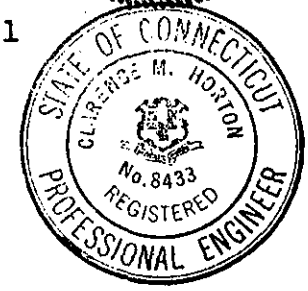
Based upon the visual inspection at the site and past performance, the dam is judged to be in poor condiiton. There are areas requiring attention, monitoring, and maintenance such as seeps along the toe of the embankment, erosion on the upstream and downstream slopes, location and operation of outlet and inlet structures and valves, and removal of trees.

It is recommended that the owner initiate further studies, to be performed by a registered professional engineer, which would include further inspection to locate inlet structures, valves, conduits, and outlet structures; investigation of seepage and wet areas at the toe of the embankment, removal of trees at the toe of the dam and in the spillway discharge channel, and preparation of "as-built" drawings for future reference. If the low-level outlet is gated on the downstream side of the dam, then measures should be taken to gate it on the upstream side to prevent pressures in the pipe within the embankment.

The above recommendations and further remedial measures presented in Section 7 should be implemented within one year of the owner's receipt of this report.



Peter M. Heynen, P.E.
Project Manager - Geotechnical
Cahn Engineers, Inc.



C. Michael Horton, P.E.
Department Head
Cahn Engineers, Inc.

This Phase I Inspection Report on Farmington Reservoir Dam (CT-00263) **has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.**

Carney M. Terzian

CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division

Richard J. DiBuono

RICHARD DIBUONO, MEMBER
Water Control Branch
Engineering Division

ARAMAST MAHTESIAN, CHAIRMAN
Geotechnical Engineering Branch
Engineering Division

APPROVAL RECOMMENDED:

Joe B. Fryar

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspection. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam would necessarily represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions will be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions there of. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as neccessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

The Phase I Investigation does not include an assessment of the need for fences, gates, no-trespassing signs, repairs to existing fences and railings and other items which may be needed to minimize trespass and provide greater security for the facility and safety to the public. An evaluation of the project for compliance with OSHA rules and regulations is also excluded.

The information contained in this report is based on the limited investigation described above and is not warranted to indicate the actual condition of the dam. The integrity of the dam can only be determined by a means of a monitoring program and/or a detailed physical investigation. The accuracy of available data is assumed where not in obvious conflict with facts observable during the visual inspection.

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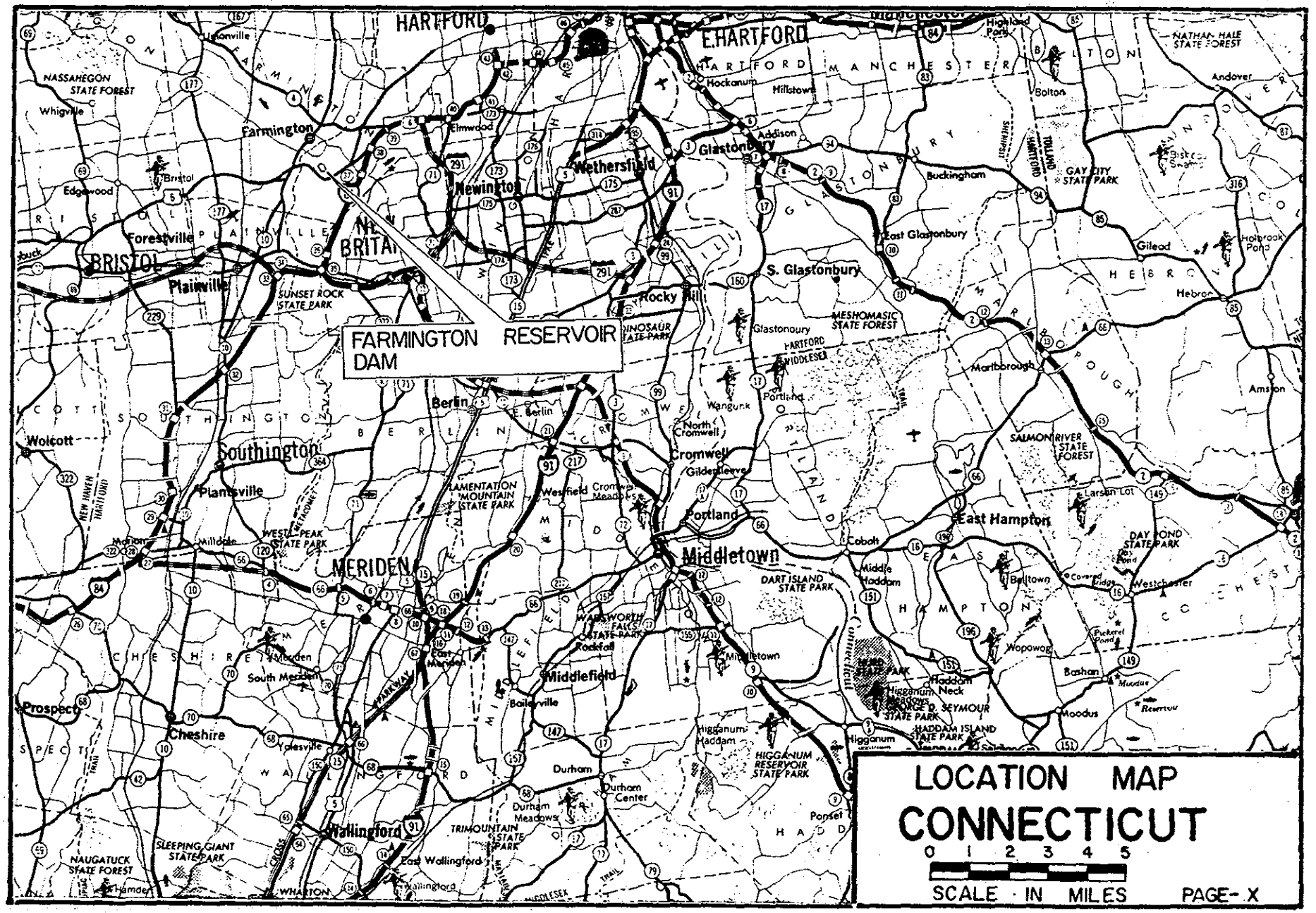
SECTION 7: ASSESSMENT, RECOMMENDATIONS & REMEDIAL MEASURES

APPENDICES

OVERVIEW PHOTO
(May, 1980)

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF	Farmington Reservoir Dam	Farmington	DATE Sept. 1980
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER	INSPECTION OF NON-FED DAMS	TR-Pequabuck River	CONNECTICUT	CE # 27785KE PAGE ix


FARMINGTON RESERVOIR DAM
LOCATION MAP
CONNECTICUT
0 1 2 3 4 5
SCALE IN MILES
PAGE- X

PHASE I INSPECTION REPORT

FARMINGTON RESERVOIR DAM

SECTION I - PROJECT INFORMATION

1.1 GENERAL

a. Authority - Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Cahn Engineers, Inc. has been retained by the New England Division to inspect and report on selected dams in the State of Connecticut. Authorization and notice to proceed were issued to Cahn Engineers, Inc. under a letter of April 14, 1980 from William E. Hodgson, Jr., Colonel, Corps of Engineers. Contract No. DACW 33-80-C-0052 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection Program - The purposes of the program are to:

1. Perform technical inspection and evaluation of non-federal dams to identify conditions requiring correction in a timely manner by non-federal interests.

2. Encourage and prepare the States to quickly initiate effective dam inspection programs for non-federal dam.

3. To update, verify and complete the National Inventory of Dams.

c. Scope of Inspection Program - The scope of this Phase I inspection report includes:

1. Gathering, reviewing and presenting all available data as can be obtained from the owners, previous owners, the state and other associated parties.

2. A field inspection of the facility detailing the visual condition of the dam, embankments and appurtenant structures.

3. Computations concerning the hydraulics and hydrology of the facility and its relationship to the calculated flood through the existing spillway.

4. An assessment of the condition of the facility and corrective measures required.

It should be noted that this report passes judgment only on those factors of safety and stability which can be determined by a visual surface examination. The inspection is to identify those visually apparent features of the dam which evidence the need for corrective action and/or further study and investigation.

1.2 DESCRIPTION OF PROJECT

a. Location - The dam is located on an unnamed tributary to the Pequabuck River (Connecticut River Basin) in a suburban area of the town of Farmington, County of Hartford, State of Connecticut. The dam is shown on the New Britain USGS Quadrangle Map having coordinates latitude N41° 42.7' and longitude W72° 49.6'.

b. Description of Dam and Appurtenances - The dam has a total length of 760 feet, has an "L shaped" configuration and is 7.7 feet in height (See Sheet B-1). The dam is of earth fill construction and is reported by the owner, to have a concrete core. The top of the dam (elevation 384.7) is 2.7 feet above the spillway crest and eight (8) feet wide, with a footpath which extends the length of the dam.

The upstream slope has an inclination of 1 horizontal to 1 vertical above the waterline and flattens out below the water, and is entirely stabilized with weeds and brush (See Photo 1, Appendix C). Riprap extends below elevation 382.7 (approximately 2 feet below the top of slope) and stabilizes the slope below the waterline. The downstream slope has cover similar to the upstream slope with large trees encroaching upon the toe. The slope inclination is 1.5 horizontal to 1 vertical.

The spillway is located at the left end of the dam. It is a 32 foot wide concrete ogee weir (crest elevation of 382.0) and apron structure with concrete training walls (See Photo 3, Sheet B-1). The approach channel is gently sloping and stabilized with riprap. The discharge channel has a hand-laid riprap lining for approximately 25 feet and is unlined thereafter. A small stone wall exists at the left side of the channel, abutting the left training wall and extending 50 feet or more.

At the toe of the dam are two sand filter basins, one dry, one containing water (See Sheet B-1, Overview Photo). The filter to the left side is 34' x 100' and the filter to the right is 74' x 100'. There is a concrete chamber located to the rear of the lefthand filter basin. This chamber is assumed to control flow from the sand filter basins. A channel extends from this concrete chamber towards the spillway discharge channel (See Sheet B-1). The owner stated that there is a 10 or 12 inch blowoff pipe, the valve of which is located at the toe of the dam near the sand filters. The outlet of this pipe could not be located although there is an 8 inch outlet near the smaller sand filter of unknown origin.

The gatehouse is located 20 feet upstream, at the central portion of the dam. It is a 10x10 foot structure with a concrete foundation and brick superstructure. It is accessible by way of a wooden bridge extending from the dam. In the gatehouse, there are 3 manually operated gate stands which are assumed to operate the

sluice gates (only two of the inlets were visible) at the upstream side of the gatehouse foundation. These sluice gates would allow water to enter a wet well from three different levels. Water in the wet well will pass through several screens before release from gatehouse. There are 3 manually operated gate stands which are assumed to operate the outlets (one of which is the gatehouse drain). The exact type, size and location of the outlet pipes and outlet valves is not known. The owner reports that these pipes lead to the filter beds at the toe of the dam.

c. Size Classification - SMALL - The dam is 7.7 feet high and impounds 64 acre-feet of water with the reservoir level at the top of the dam. According to the Recommended Guidelines a dam with this height and maximum impoundment capacity is classified as small in size.

d. Hazard Classification - HIGH - If the dam were breached, there is potential for loss of more than a few lives as well as substantial property damage. At least two houses on Dorset Lane, 2000 ft. downstream of the dam would be flooded with 3.5 and 5 feet of water respectively. The flood would wash out culverts at Dorset Lane and Reservoir Road.

e. Ownership - The Farmington Water Company
105 Main St.
Farmington, Conn. 06032
Mr. Arthur Deming, President
(203) 677-1571
Mr. William Wadsworth, Owner
(203) 677-1870

f. Operator - Same as above.

g. Purpose of Dam - The dam was constructed to impound water for supply to the Town of Farmington. At the present time, the dam has been abandoned as a water supply facility, and has no known functional purpose.

h. Design and Construction History - According to the owner, there are no known engineering plans of the as built structure. The valves, intakes, and treatment facilities were designed by Mr. Hill, a professional engineer from New Haven, Connecticut in 1926. The location of these plans is not known.

According to the owner, the dam was first constructed by Adrian Wadsworth, the owner's father, and founder of the Farmington Water Company in 1895. The impoundment was "spring fed". Additions to the dam were constructed in 1910, 1918, and 1930. In 1930, the dam was raised and a concrete spillway was added.

The dam was operated until 1973 when water quality became unacceptable due to algae problems and inadequate treatment capability. It has not been operated since.

i. Normal Operation Procedures - The reservoir is no longer used as a water supply facility. All valves are reported to be closed. The natural flow leaves the reservoir level at the spillway crest. The water company inspects the area informally approximately one time per month. No formal operation records or lake level readings are known to exist.

1.3 PERTINENT DATA

a. Drainage Area - 0.26 square miles of rolling to mountainous terrain in the Connecticut River Basin, of which less than half is wooded. A housing development is present in the southwestern portion of the watershed.

b. Discharge at Damsite - Normal discharge is over the spillway.

1. Outlet Works (conduits):	Not known
2. Maximum flood at damsite:	Not Known
3. Ungated spillway capacity @ top of dam el. 384.7:	525 cfs
4. Ungated spillway capacity @ test flood el. 383.7:	250 cfs
5. Gated spillway capacity @ normal pool:	N/A
6. Gated spillway capacity @ test flood:	N/A
7. Total spillway capacity @ test flood el. 383.7:	250 cfs
8. Total project discharge @ test flood el. 383.7:	250 cfs

c. Elevations - All elevations are NGVD based on an assumed spillway elevation, See Sheet B-1).

1. Toe of dam:	377± (varies)
2. Bottom of cutoff:	N/A
3. Maximum tailwater:	N/A
4. Normal pool:	382.0

5. Full flood control pool:	N/A
6. Spillway crest (ungated):	382.0
7. Design surcharge (original design):	Unknown
8. Top of dam:	384.7
9. Test flood surcharge:	383.7

d. Reservoir Length (feet)

1. Normal pool:	2000 ft.
2. Flood control pool:	N/A
3. Spillway crest pool:	2000 ft.
4. Top of dam:	2100 ft.
5. Test flood pool:	2100 ft.

e. Reservoir Storage (acre-ft.)

1. Normal pool:	23.5 acre-ft.
2. Flood control pool:	N/A acre-ft.
3. Spillway crest pool:	23.5 acre-ft.
4. Top of dam pool:	64 acre-ft.
5. Test flood pool:	52 acre-ft.

f. Reservoir Surface

1. Normal pool:	14 acres
2. Flood control pool:	N/A
3. Spillway crest pool:	14 acres
4. Top of dam pool:	16 acres
5. Test flood pool:	15.5 acres

g. Dam

1. Type:	Earth embankment
2. Length:	760 ft.
3. Height:	7.7 ft.

4. Top width:	8 ft.
5. Side slopes:	1H to 1V Upstream (above waterline) 1.5H to 1V Downstream
6. Zoning:	N/A
7. Impervious core:	Unknown (reported to be concrete by owner)
8. Cutoff:	Not Known
9. Grout curtain:	Not Known
10. Other:	N/A

h. Diversion and Regulating Tunnel - N/A

i. Spillway

1. Type:	Concrete, ogee weir
2. Length of weir:	32 ft.
3. Crest elevation:	382.0
4. Gates:	N/A
5. Upstream channel:	Riprap
6. Downstream channel:	Riprap
7. General:	N/A

j. Regulating Outlets - There is no available information regarding outlets.

SECTION 2: ENGINEERING DATA

2.1 DESIGN DATA

According to the operator, plans of the treatment works including intake structures and valves were prepared in 1926 by an engineer named Hill, from New Haven, Connecticut. These plans have not been located. There are no engineering values, assumptions, test results or calculations available for the original construction or subsequent dam raisings and spillway construction.

Design features are described on the basis of field inspection and information reported by the owner in Section 1.2 (b) of this report.

2.2 CONSTRUCTION DATA

There is no written data available for the original construction of the dam. The owner feels that no plans were prepared for the original dam, but reports that the dam and subsequent additions in 1910, 1918 and 1930 were constructed by the original owner, Mr. Adrian Wadsworth. Likewise, there are no plans for subsequent raisings of the dam.

2.3 OPERATION DATA

The dam is no longer in operation. No formal operational records are known to exist. Lake level readings are not made. The owner provided the following information:

1. The dam has never been overtopped or breached.

2. The reservoir capacity at one time was 7 million gallons.

3. When operating, a daily usage of 30,000 cu. ft. was more than the reservoir could sustain.

4. The dam was last operated in 1973.

2.4 EVALUATION

a. Availability - Existing data was provided by the State of Connecticut and by the owner, who made the premises available for visual inspection.

b. Adequacy - The limited amount of engineering data available is inadequate to perform an in-depth assessment of the dam, therefore, the assessment of this dam must be based on visual inspection, hydraulic computations, hydrologic judgements, and information provided verbally by the owner.

c. Validity - A comparison of the available information and visual observations reveals some discrepancies relative to dam dimensions as recorded in the State of Connecticut's Dam Inventory data sheet.

SECTION 3: VISUAL INSPECTION

3.1 FINDINGS

a. General - Based upon the visual inspection performed on May 12, 1980 the condition of the dam is poor. Inspection revealed problem areas requiring various levels of maintenance, monitoring and repair. The reservoir was at elevation 382.2 at the time of the inspection.

b. Dam

Top of Dam - The top of the dam is vegetated with grass and brush (Photo 1). A footpath traverses its length. An area of erosion was noted to the right of the gatehouse access bridge (Photo 5).

Upstream Slope - The upstream slope is steep, shows evidence of erosion and needs more riprap protection. A large eroded area extends from the top of dam down the upstream face near (to the right of) the gatehouse (Photo 5). The upstream slope is vegetated with weeds and brush.

Downstream Slope - The downstream slope is being encroached with large trees and heavy brush (Photos 1 and 2). It is irregular and eroded in places. There are areas of stagnant water and small seeps at the toe of the dam (Photo 6).

Spillway - The right training wall has some deterioration at the downstream end, exposing the aggregate (Photo 4). The concrete portion of the left training wall is in good condition, but a small downstream stone wall is broken up. The approach channel is riprapped near the spillway, but has brush and saplings growing farther out in the reservoir along the left side. The discharge channel has a stone bottom, some of which has been washed out.

c. Appurtenant Structures

Intake Structure (Gatehouse) - The exterior foundation and walls were in good condition, however the interior of the gatehouse and the door require maintenance and minor repair.

d. Reservoir Area - The area surrounding the reservoir is rolling, approximately 1/3 wooded, the remainder open land. The watershed is bisected by State Route 6. Reservoir Road approximates the northwest boundary of the watershed.

e. Downstream Channel - The downstream channel is heavily vegetated with overhanging trees and debris in the channel. A single channel flows beneath Reservoir Rd. and discharges to the Pequabuck River 1.5 miles downstream of the dam. A small pond is located approximately 2000 ft. downstream of the dam.

3.2 EVALUATION

Based upon the visual inspection, this dam is assessed as being in poor condition. The following features which could influence the future condition and/or stability of the dam were identified.

1. The erosion area located at the top of the dam and extending down the upstream slope to the right of the gatehouse will erode should the dam become overtopped.

2. Growth of trees and heavy brush on the toe and downstream slope can promote piping and/or seepage by creating flow paths along root systems in the embankment. Large trees, if uprooted may produce depressions in the embankment which may be critical to the stability of the dam.

3. Areas of seepage and standing water at the toe of the slope could result in future stability problems if flows increase without detection.

4. The lack of proper operating and maintenance procedures and information concerning low-level outlets for lowering the reservoir, leaves the dam more susceptible to failure, as well as the owner unprepared, during high flooding condiitons.

5. Trees, brush and debris in spillway discharge channel will impede flows during periods of high project discharge.

6. The steep slope and lack of proper protection on the upstream side of the embankment does not provide adequate safety against sloughing and regional slope failure.

SECTION 4: OPERATIONAL AND MAINTENANCE PROCEDURES

4.1 OPERATING PROCEDURES

a. General - No formal operation procedures are known to exist. Lake level readings are not taken at the dam. The low-level outlet has not been operated "in some years", as reported by the operator and could not be located during the inspection.

b. Description of Any Warning System in Effect - No formal downstream warning system is known to be in effect.

4.2 MAINTENANCE PROCEDURES

a. General - No formal maintenance procedures are known to exist. The brush is cut from the slopes once a year.

b. Operating Facilities - No maintenance is performed for the operating facilities.

4.3 EVALUATION

The operation and maintenance procedures are poor. A formal program of operation and maintenance procedures should be implemented by the owner, including documentation to provide complete records for future reference. Also, a formal emergency action plan and downstream warning system should be developed and implemented within the time period indicated in Section 7.1c. Remedial operation and maintenance recommendations are presented in Section 7.

SECTION 5: EVALUATION OF HYDRAULIC/HYDROLOGIC FEATURES

5.1 GENERAL

The watershed is 0.26 sq. mi. of rolling to mountainous terrain located in the Connecticut River Basin, of which less than half is wooded. A housing development exists in the south-western portion of the watershed and State Route 6 runs through the central portion of the watershed. The maximum impoundment to the top of the dam (El. 384.7 NGVD) is estimated to be 64 ac. ft. and estimated storage below spillway crest is 23.5 ac. ft.

The dam is classified as being small in size and having a high hazard classification.

5.2 DESIGN DATA

No hydraulic or hydrologic design data are available for this dam.

5.3 EXPERIENCE DATA

No information on any serious problem situations arising at the dam or downstream reaches of the dam was found. The maximum previous discharge at this dam is unknown.

5.4 TEST FLOOD ANALYSIS

The test flood for this high hazard, small size dam is in the half Probable Maximum Flood (½ PMF) to PMF range. Selecting ½ PMF as test flood for this dam based on the size of the dam and involved downstream risk potential, the Corps of Engineers recommended guidelines for drainage areas below 2 sq. mi. (rolling to mountainous terrain) yields a peak inflow of 315 cfs at the test flood for an estimated PMF of 2400 cfs per square mile. The peak outflow is estimated to be 250 cfs with the maximum stage in the reservoir at 383.7 NGVD and maximum surcharge above the spillway crest is estimated to be 1.7 feet. Thus, the dam is not expected to overtop at the selected test flood conditions. The spillway capacity with pool at top of dam is estimated to be 525 cfs which is greater than 100% of the routed test flood outflow. Computations for conditions at the full PMF have also been performed and are given on page D-27 in Appendix D.

5.5 DAM FAILURE ANALYSIS

Utilizing the Corps of Engineers April 1978 "Rule of Thumb Guidance for Estimating Downstream Failure Hydrographs", the peak failure outflow due to dam breach is estimated to be 9200 cfs with an estimated flood depth of 3.5 feet immediately downstream of the dam. The flood routing was performed for peak failure outflow with pool at top of dam. The prefailure flow in the stream is estimated

to be 525 cfs causing depths of 3.1 feet and 1.7 feet in the stream bed at initial and second impact areas respectively. After failure the flood stage is estimated to increase by 4.4 ft. at the initial impact area and by 2.5 ft. at the second impact area thereby resulting in the loss of more than a few lives.

The estimated peak flow rates and peak flood depths at five sections downstream of the dam resulting from a dam failure are:

D/S Section (Ft. from Dam)	Flow (CFS)	Flood Depth (Ft.)	Velocity FPS	Volume Remaining Ac. Ft.
At Dam	9200	3.5	-	64
200	7000	5.4	4.2	49
375	7000	6.8	13	49
1825	6300	7.5	15	44
2125	5600	4.6	8.5	39
2375	4800	4.2	5	33

A flood of this magnitude would flood at least two houses on Dorset Lane housing development 2000± feet downstream of the dam. The first floor of the house located 4± feet above the streambed, (east of Dorset Lane adjacent to section DD) would be flooded with 3.5± feet of water. The velocity of flood water in the vicinity of this house is estimated to be 15 FPS, which could cause severe damage to the structures including the culvert near this house. This potential damage area is designated initial impact area and shown as such on Sheet D-1. Further downstream another house located in between Dorset Lane and a small pond would be flooded with 5± feet of water (second impact area on Sheet D-1). In addition, Reservoir Road, downstream of the dam would be inundated and the culvert at this road would be damaged. Also, within the 2374 feet reach only 48% of the flood volume is expected to be attenuated (Appendix D-25 & 26).

Based on the hydraulic/hydrologic analysis and the potential for loss of more than a few lives, the dam has a high hazard classification.

SECTION 6: EVALUATION OF STRUCTURAL STABILITY

6.1 VISUAL OBSERVATIONS

The dam is an earth embankment with a 32 foot wide concrete spillway at the left end. The dam is 7.7 feet high and 8 feet wide at the top. The upstream slope is 1H to 1V above waterline and slopes out below the waterline. The downstream slope is 1.5H to 1V. The dam is reported to have a concrete core but no information could be found to support this. The whole embankment has a weed and brush cover. Small seeps and wet areas were noted at the toe of the dam. The slopes are quite irregular with a large erosion area (from trespassing) on the upstream slope just right of the gatehouse. The low-level outlet could not be located during the inspection.

Recommendations addressing these items and other remedial measures are presented in Section 7.

6.2 DESIGN AND CONSTRUCTION DATA

No information is available.

6.3 POST CONSTRUCTION CHANGES

Post construction changes include lengthening the dam through the years between 1910 and 1930, raising the embankment 2-3 feet and adding the concrete spillway in 1930.

6.4 SEISMIC STABILITY

The dam is in Seismic Zone 1, and according to the Army Corps of Engineers Recommended guidelines, need not be evaluated for seismic stability.

SECTION 7: ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Condition - Based upon the visual inspection of the site and past performance, the dam appears to be in poor condition. There are a number of areas of concern which require maintenance, repair and monitoring. These include seeps at the toe, erosion areas, woody vegetation on the downstream slope, and the lack of information on the outlet location and operation.

Based upon "Preliminary Guidance for Estimating Maximum Probable Discharge" dated March, 1978 and hydraulic/hydrologic computations, peak inflow to the reservoir at the ½ PMF is 315 cubic feet per second; peak outflow is 250 cubic feet per second with the dam retaining 1.0 feet of freeboard. The spillway capacity to the top of dam is 525 cubic feet per second, which is greater than 100% of the routed test flood outflow.

b. Adequacy of Information - The information available is such that an assessment of the condition and stability of the dam must be based solely on visual inspection, past performance of the dam, and sound engineering judgement.

c. Urgency - It is recommended that the measures presented in Section 7.2 and 7.3 be implemented within 1 year of the owner's receipt of this report.

7.2 RECOMMENDATIONS

It is recommended that further studies be made by a registered professional engineer qualified in dam design and inspection pertaining to the following:

1. The origin and significance of seepage and wet areas located at the toe of the embankment.

2. Development of a program for monitoring seepage along the toe of the dam.

3. The low-level outlet should be located and inspected. This should include its operation and a check of the outlet channel. If the outlet valve is located on the downstream slope, measures should be taken to gate the outlet on the upstream side of the dam so as to eliminate pressures in the pipe within the embankment.

4. The gatehouse should be inspected to check the exact location and function of valves, inlets and conduits to determine their use as related to the filter basins or low-level outlet.

5. Preparation of "as built" drawings for future reference.

6. Large trees encroaching upon the dowstream slope should be removed, backfilled and proper slope protection placed.

7. The upstream slope should be graded to a slope of 2 horizontal to 1 vertical or flatter and slope protection placed to well above the normal waterline.

7.3 REMEDIAL MEASURES

a. Operation and Maintenance Procedures - The following measures should be undertaken within the time period indicated in Section 7.1c, and continued on a regular basis.

1. Round-the-clock surveillance should be provided by the owner during periods of heavy precipitation and high project discharge. The owner should develop and implement an emergency action plan and downstream warning system in case of emergencies at the dam.

2. A formal program of operation and maintenance procedures should be instituted and fully documented to provide accurate records for future reference. As part of the maintenance procedures, the owner or owner representative should perform documented monthly inspections.

3. A comprehensive program of inspection by a registered professional engineer qualified in dam design and inspection should be instituted on a biennial basis.

4. Erosion from trespassing on the slopes and top of dam should be filled and proper protection placed.

5. All trees, brush and debris should be removed from the spillway and along the spillway discharge channel to Reservoir Road.

6. Small trees and sumac on the slopes should be removed.

7. The gatehouse should be restored to a more accessible condition including repairs to the access ramp.

8. Deteriorated concrete at the spillway right training wall should be repaired.

7.4 ALTERNATIVES

1) Drain the reservoir and remove the dam.

APPENDIX A

INSPECTION CHECKLIST

VISUAL INSPECTION CHECK LIST

PARTY ORGANIZATION

PROJECT Farmington Reservoir Dam

DATE: May 12, 1980

TIME: 11:00 AM - 12:30 PM

WEATHER: Cloudy - 60°F

W.S. ELEV. 382.2 U.S.

_____ U.S.

PARTY:	INITIALS:	DISCIPLINE:
1. Peter M. Heynen	PMH	Cahn - Geotechnical
2. Miron Petrovsky	MP	Cahn - Geotechnical
3. Murali Atluru	MA	DTC - Hydrologic
4. Jay A. Costello	JAC	Cahn - Geotechnical
5. Jeffrey O. Borne	JOB	Cahn - Geotechnical
6. Tim K. Kavanaugh	TK	Cahn - Survey

	PROJECT FEATURE	INSPECTED BY	REMARKS
1.	Dam Embankment	PMH, MP, JAC, MA, JOB, TK	A-2
2.	Spillway	PMH, MP, JAC, MA, TK	A-3
3.			
4.			
5.			
6.			
7.			
8.			
9.			
10.			
11.			
12.			

PERIODIC INSPECTION CHECK LIST

PROJECT Farmington Reservoir Dam DATE May 12, 1980

PROJECT FEATURE Embankment BY JAS, PMH, MP, MA, JOB, TK

AREA EVALUATED	CONDITION
DAM EMBANKMENT	
Crest Elevation	384.7
Current Pool Elevation	382.2
Maximum Impoundment to Date	Unknown
Surface Cracks	None observed
Pavement Condition	N/A
Movement or Settlement of Crest	None observed
Lateral Movement	
Vertical Alignment	Appears good
Horizontal Alignment	
Condition at Abutment and at Concrete Structures	
Indications of Movement of Structural Items on Slopes	None observed
Trespassing on Slopes	Footpath along crest
Sloughing or Erosion of Slopes or Abutments	Slope erosion in area of gatehouse ramp
Rock Slope Protection-Riprap Failures	Some displacement
Unusual Movement or Cracking at or Near Toes	None observed
Unusual Embankment or Downstream Seepage	Some seepage and wet areas along toe
Piping or Boils	None observed
Foundation Drainage Features	
Toe Drains	
Instrumentation System	

PERIODIC INSPECTION CHECK LIST

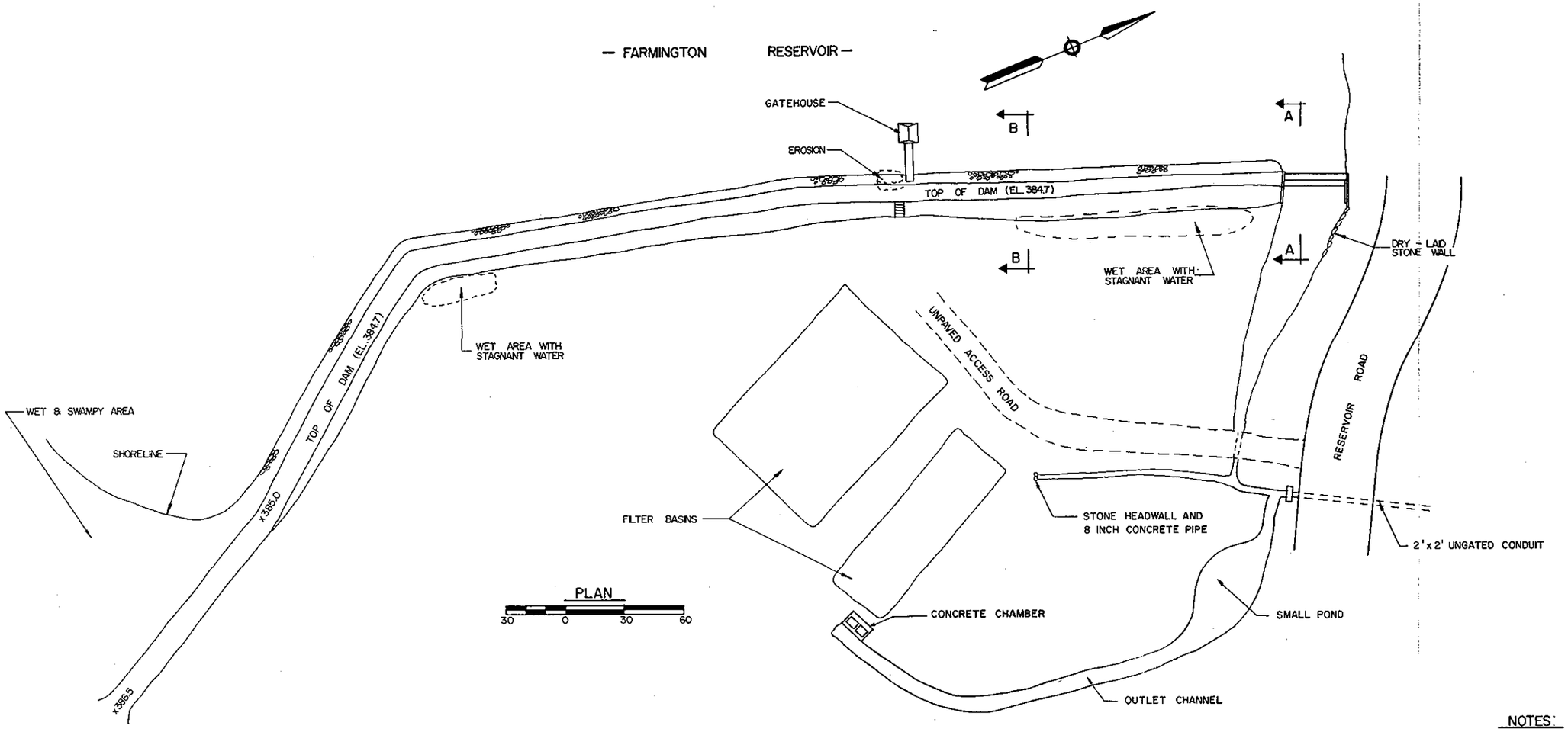
PROJECT Farmington Reservoir Dam DATE May 12, 1980

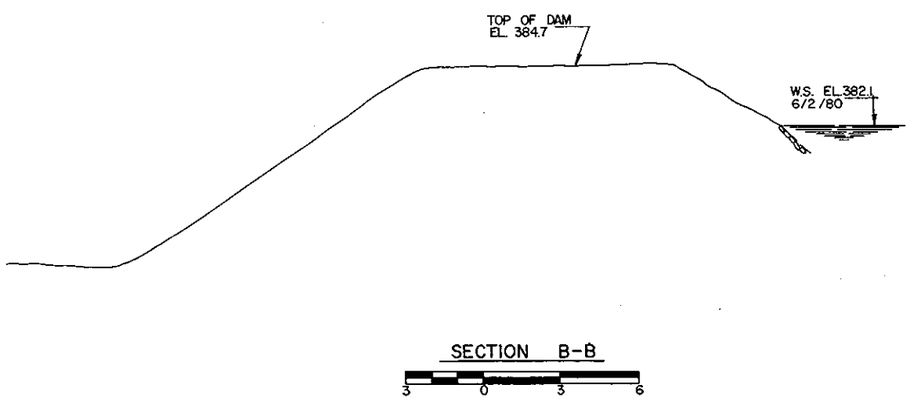
PROJECT FEATURE Spillway BY PMH, JAC, MPM, TK

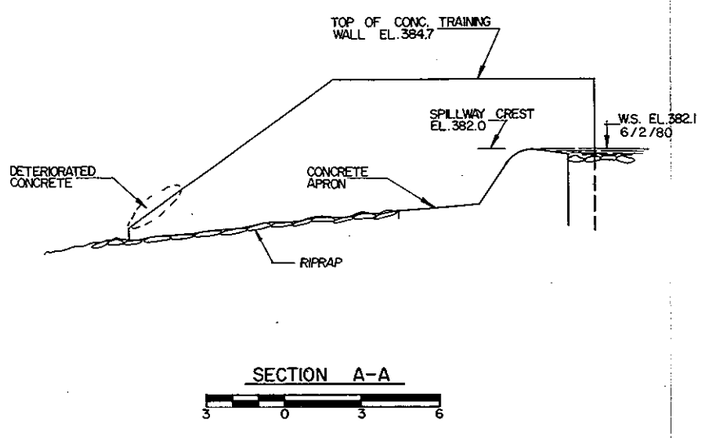
AREA EVALUATED	CONDITION
OUTLET WORKS-SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS	
a) Approach Channel	
General Condition	Riprap appears good, brush & small trees growing in channel
Loose Rock Overhanging Channel	N/A
Trees Overhanging Channel	N/A
Floor of Approach Channel	Good
b) Weir and Training Walls	
General Condition of Concrete	Some slight spalling, area of cracking & deterioration on rightwall
Rust or Staining	None observed
Spalling	Right training wall
Any Visible Reinforcing	None observed
Any Seepage of Efflorescence	None observed
Drain Holes	None observed
c) Discharge Channel	
General Condition	Poor
Loose Rock Overhanging Channel	None observed
Trees Overhanging Channel	Yes
Floor of Channel	Loose riprap, debris, boulders
Other Obstructions	N/A

APPENDIX B

ENGINEERING DATA AND CORRESPONDENCE







NOTES:

1. THIS PLAN WAS COMPILED FROM CAHN ENGINEERS INSPECTION OF THE DAM DAM DATED JUNE 2, 1980. DIMENSIONS SHOWN ARE APPROXIMATE. NOT ALL TOPOGRAPHIC AND OR STRUCTURAL FEATURES ARE NECESSARILY IDENTIFIED.
2. NO ELEVATIONS WERE AVAILABLE FOR THE DAM THEREFORE THE WATER SURFACE ELEVATION OF 382.0 FOR THE RESERVOIR SHOWN ON THE 1972 U.S.G.S. NEW BRITAIN QUADRANGLE MAP WAS ASSUMED TO BE THE N.G.V.D. ELEVATION OF THE SPILLWAY CREST. ALL OTHER ELEVATIONS SHOWN ARE REFERENCED TO THE ASSUMED SPILLWAY CREST ELEVATION.

CAHN ENGINEERS INC. WALLINGFORD, CONNECTICUT ENGINEER	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

PLAN & SECTIONS

FARMINGTON RESERVOIR DAM

TR- PEQUABUCK RIVER FARMINGTON, CONNECTICUT

DRAWN BY	CHECKED BY	APPROVED BY	SCALE: AS NOTED	
M. Norman	JAC	[illegible]	DATE: SEPT. 1980	SHEET 8-1

FARMINGTON RESERVOIR DAM

EXISTING PLANS

None Available

SUMMARY OF DATA AND CORRESPONDENCE

DATE	TO	FROM	SUBJECT	PAGE
No Date	File	State Board of the Supervision of Dams	Inventory Data	B-3

No. ~~F-7~~ 2

WATER RESOURCES COMMISSION
SUPERVISION OF DAMS
INVENTORY DATA

Long 72-49.6
Lat 41-42.7

Inventoried By J.C.

Date 6-13 K8

Name of Dam or Pond Farmington Reservoir

Code No. F 0.75 U 1.0

Nearest Street Location Reservoir Road

Town Farmington #2 Pumping House

U.S.G.S. Quad. New Britain

Name of Stream Trib of Pequabuck River

Owner Town OF Farmington ok 6/77

Address _____

Farm Water Co

1895 - 1910 - 1929 (677-1870)

Pond Used For Public Water Supply DA 0.35 sm

Dimensions of Pond: Width _____ Length _____ Area 13.8 Fasano

Total Length of Dam 250' (760') Length of Spillway 32'

Location of Spillway _____

Height of Pond Above Stream Bed 15'

Height of Embankment Above Spillway 3.6'

Type of Spillway Construction No Spillway, drawdown Pipe

Type of Dike Construction Earth

Downstream Conditions Road 200' below - Houses ½ mile below

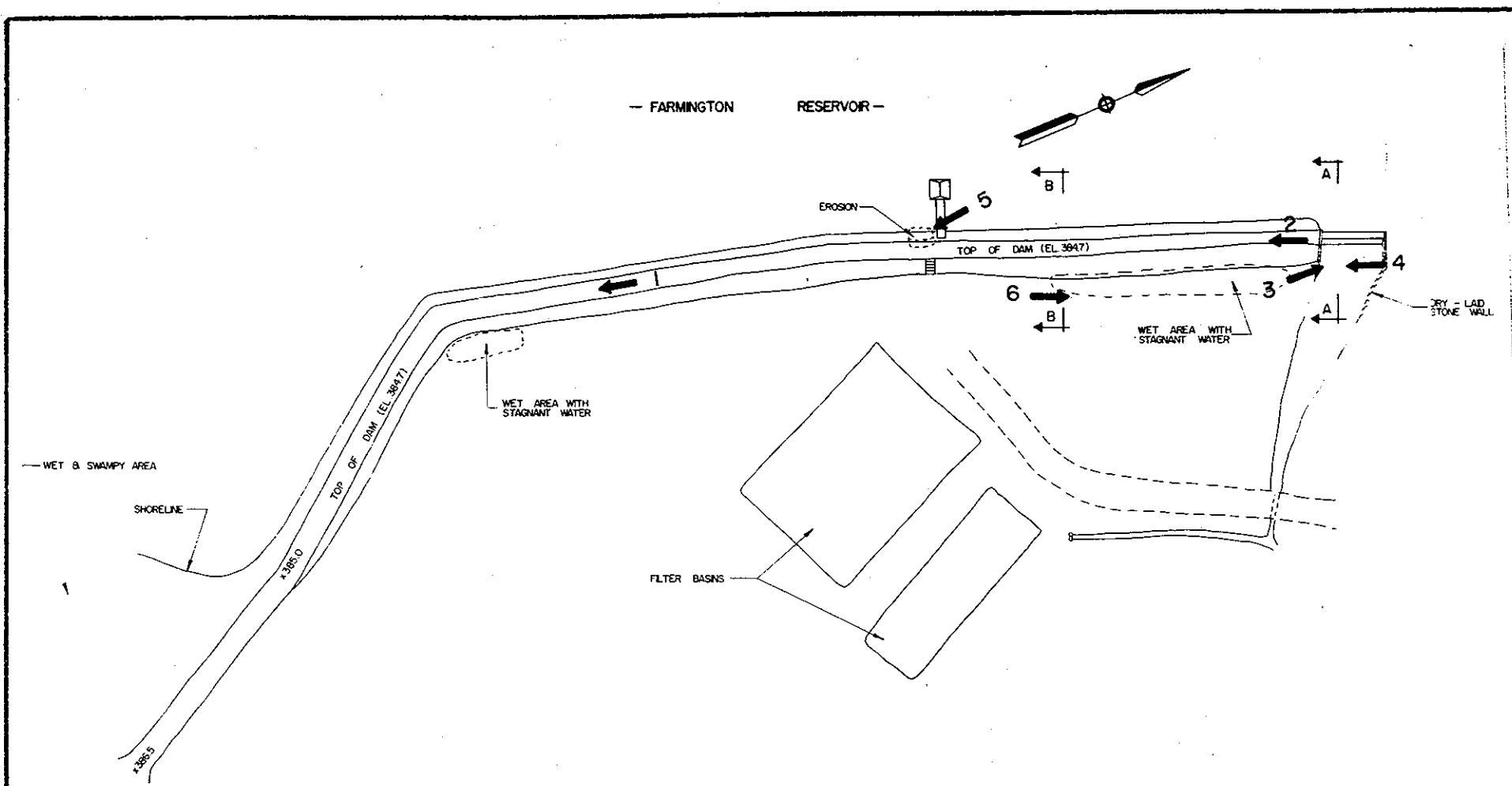
Summary of File Data _____

Remarks _____

Class B

APPENDIX C

DETAIL PHOTOGRAPHS



2 PHOTO NUMBER AND DIRECTION

PHOTO LOCATION PLAN

FARMINGTON RESERVOIR DAM

SHEET C-1

Photo 1 - Upstream slope from gate house (May, 1980)

Photo 2 - Top of dam and downstream slope from spillway. Gate house in background, (May, 1980).

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF	Farmington Reservoir Dam TR-Pequabuck River Farmington, CT
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER	INSPECTION OF NON-FED. DAMS	CE # 27785KE DATE Sept. 1980 PAGE C-1

Photo 3 - Ogee shaped concrete weir at left end of dam, (May, 1980).

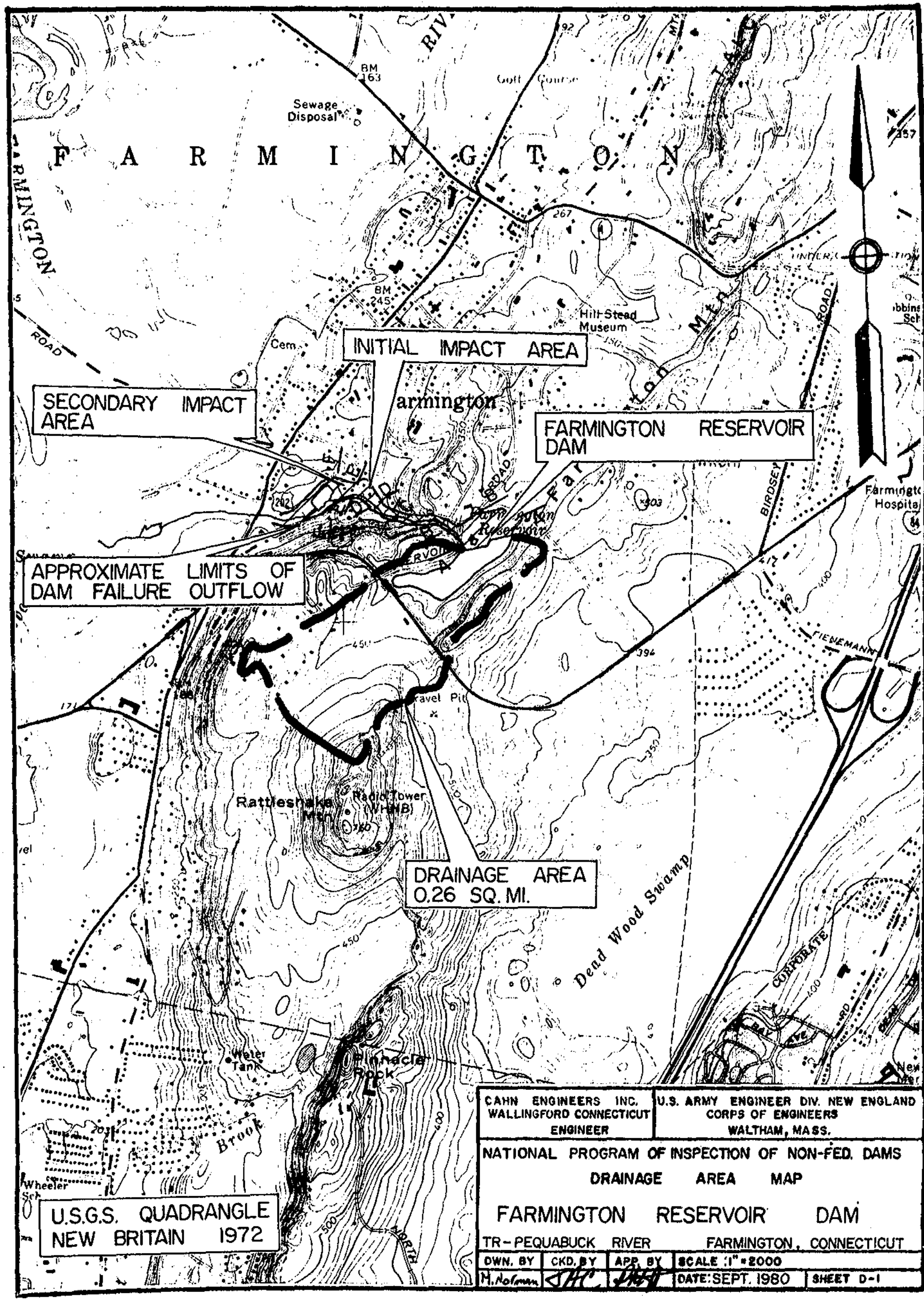
Photo 4 - Deterioration of concrete at right spillway wall, (May, 1980).

Photo 5 - Erosion on upstream slope just to the right of the gate house, (May, 1980).

Photo 6 - Wet area with stagnant pool along toe of dam, (May, 1980).

APPENDIX D

HYDRAULICS/HYDROLOGIC COMPUTATIONS


FARMINGTON
INITIAL IMPACT AREA
SECONDARY IMPACT AREA
FARMINGTON RESERVOIR DAM
APPROXIMATE LIMITS OF DAM FAILURE OUTFLOW
DRAINAGE AREA 0.26 SQ. MI.
Rattlesnake Mtn
Radio Tower (WHNB)
Pinnacle Rock
Dead Wood Swamp
Hill-Stead Museum
Sewage Disposal
Golf Course
Gravel Pit
Farmington Hospital
Water Tank
Brook
Wheeler Sch
U.S.G.S. QUADRANGLE NEW BRITAIN 1972
CAHN ENGINEERS INC. WALLINGFORD CONNECTICUT ENGINEER
U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS
DRAINAGE AREA MAP
FARMINGTON RESERVOIR DAM
TR-PEQUABUCK RIVER
FARMINGTON, CONNECTICUT
DWN. BY
CKD. BY
APP. BY
SCALE 1" = 2000
DATE: SEPT. 1980
SHEET D-1

PERFORMANCE AT PEAK FLOOD CONDITIONS

PROBABLE MAXIMUM FLOOD (PMF) DETERMINATION—

DRAINAGE AREA - 0.26 SQ. MI. PLANIMETERED FROM USGS MAP.

WATERSHED CLASSIFICATION — "ROLLING" TO "MOUNTAINOUS" BASED ON USGS MAP AND SITE VISIT.

PMF PEAK INFLOW:
FOR DRAINAGE AREA < 2 SQ. MI. WITH ROLLING TO MOUNTAINOUS TERRAIN, THE CORPS OF ENGINEERS RECOMMENDS CSM VALUE TO BE IN THE 2000 TO 2500 CFS/SQ. MI. RANGE.
ACCORDINGLY, FOR THE FARMINGTON RESERVOIR WATERSHED, PEAK FLOW RATE = 2400 CFS/SQ. MI. IS SELECTED.
PMF PEAK INFLOW: $2400 \times 0.26 = 625$ CFS
½ PMF PEAK INFLOW ≅ 315 CFS

SIZE CLASSIFICATION—
FOR THE PURPOSE OF DETERMINING PROJECT SIZE, THE MAXIMUM STORAGE ELEVATION IS CONSIDERED EQUAL TO THE TOP OF DAM.
TOP OF DAM = EL. 384.7 * NGVD
HEIGHT OF DAM = EL. 384.7 − EL. 377.0 (TOE OF THE DAM)
= 7.7 FT

* THE W.S ELEVATION 382 MSL ON THE NEWBRITAIN, CT. QUADRANGLE SHEET (REV. 1972) IS ASSUMED TO BE THE SPILLWAY CREST ELVN. ON NATIONAL GEODETIC VERTICAL DATUM (NGVD). ALL OTHER ELEVATIONS ARE REFERENCED TO THIS ASSUMED ELVN. AND ARE OBTAINED BASED UPON INFORMATION FURNISHED BY CAHN ENGINEERS.

PLANIMETERING FROM USGS MAP FOR RESERVOIR SURFACE AREAS

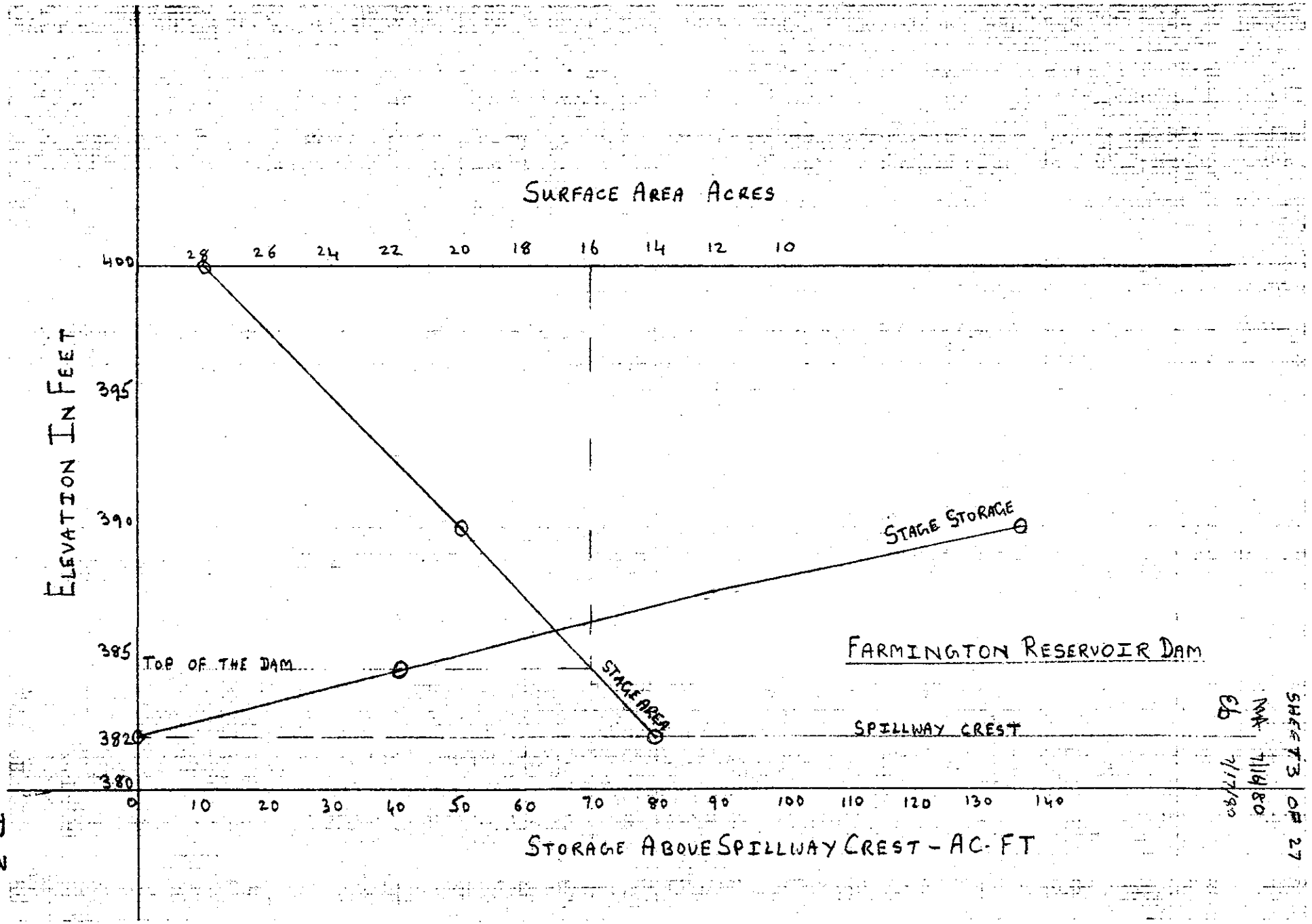
AT EL. 382 (SPILLWAY CREST) = 14 ACRES
AT EL. 390 = 20 ACRES
AT EL. 400 = 28 ACRES

A STAGE-RESERVOIR AREA CURVE IS PLOTTED (SHEET 3)
FROM THIS CURVE, RESERVOIR AREA AT TOP OF DAM = 16 ACRES.
AVERAGE RESERVOIR AREA BETWEEN SPILLWAY CREST AND TOP OF DAM = 15 ACRES
STORAGE BETWEEN SPILLWAY CREST & TOP OF DAM
$= 2.7 \times 15 = 40.5$ AC.FT.

ESTIMATED STORAGE BELOW SPILLWAY CREST $= \frac{1}{3}bh$
$= \frac{1}{3} \times 14 \times 5 \cong 23.5$ AC.FT.
($b = 14$ AC. $h =$ EL 382 − EL 377 $= 5'$)

∴ MAXIMUM IMPOUNDMENT TO TOP OF DAM
$= 40.5 + 23.5 = 64$ AC.FT.

A STAGE-STORAGE CURVE IS PLOTTED ON SHEET 3
THUS, ACCORDING TO CORPS OF ENGINEERS GUIDELINES TABLE 1, THE FARMINGTON RESERVOIR DAM IS CLASSIFIED AS SMALL BASED UPON THE STORAGE CAPACITY OF 64 AC. FT (< 1000 AND ≥ 50) AND HEIGHT OF THE DAM IS ONLY 7.7'.


SURFACE AREA ACRES
28
26
24
22
20
18
16
14
12
10
400
395
390
385
382
380
ELEVATION IN FEET
STAGE STORAGE
TOP OF THE DAM
STAGE AREA
SPILLWAY CREST
FARMINGTON RESERVOIR DAM
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
STORAGE ABOVE SPILLWAY CREST - AC-FT

D-3

HAZARD POTENTIAL - HIGH HAZARD POTENTIAL DAM BASED ON DAM BREACH ANALYSIS AND RELATIVE LOCATIONS OF HOUSES AND OTHER STRUCTURES. A DETAILED DISCUSSION OF FAILURE HAZARD POTENTIAL IS INCLUDED AT THE END OF BREACH ANALYSIS SECTION OF APPENDIX - D

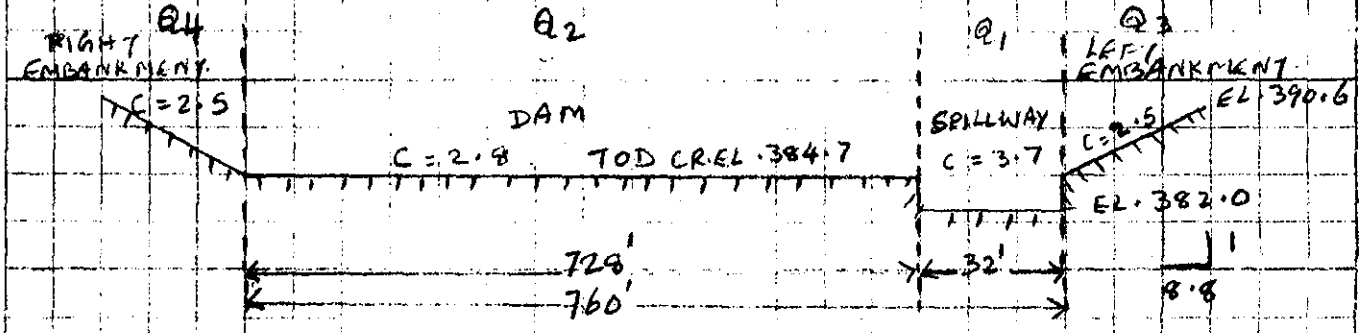
SELECTION OF TEST FLOOD -
FOR THE SMALL SIZE AND HIGH HAZARD POTENTIAL CLASSIFICATION, TABLE 3 OF CORPS OF ENGINEERS RECOMMENDED GUIDELINES. THE TEST FLOOD COULD BE IN THE ½PMF TO PMF RANGE. BECAUSE OF THE SMALL SIZE OF THE PROJECT (THE DAM IS ONLY 7.7 FT HIGH),

A TEST FLOOD IN THE LOWER RANGE IS SELECTED. TEST FLOOD = ½PMF

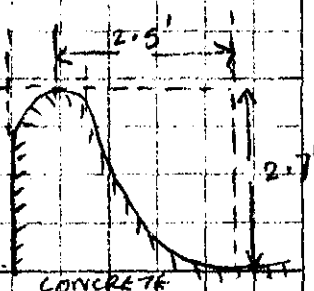
TEST FLOOD PEAK INFLOW = 315 CFS

NOTE: SURCHARGE STORAGE ROUTING IS ALSO PERFORMED FOR PMF PEAK INFLOW.

COMPOSITE DISCHARGE RATING CURVE



POTENTIAL OVERFLOW PROFILE
(BASED ON CE FIELD INFORMATION)



SPILLWAY –

$Q_1 = CLH^{3/2}$

FOR SPILLWAY WITH A ROUND CRESTED SHAPE C=3.7 ASSUMED (REF: HANDBOOK OF APPLIED HYDROLOGY BY VEN TE CHOW CHAPTER 7)

$= 3.7 \times 32 \times H^{3/2}$

$= \underline{118.4 H^{3/2}}$

DAM –

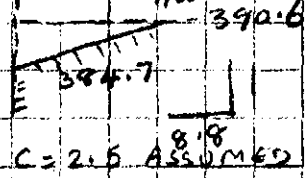
$Q_2 = CLH^{3/2}$ C = 2.8 ASSUMED (CREST)

L = 728'

CREL 384.7

$= \underline{2038 H^{3/2}}$

LEFT EMBANKMENT –



$Q_3 = \frac{2}{5} CL (hb - ha)^{5/2}$ *

$= \underline{8.8\, hb^{5/2}}$ FOR ha = 0 UP TO EL 390.6

RIGHT EMBANKMENT – DISCHARGE COMPUTED SIMILAR TO LEFT EMBANKMENT.

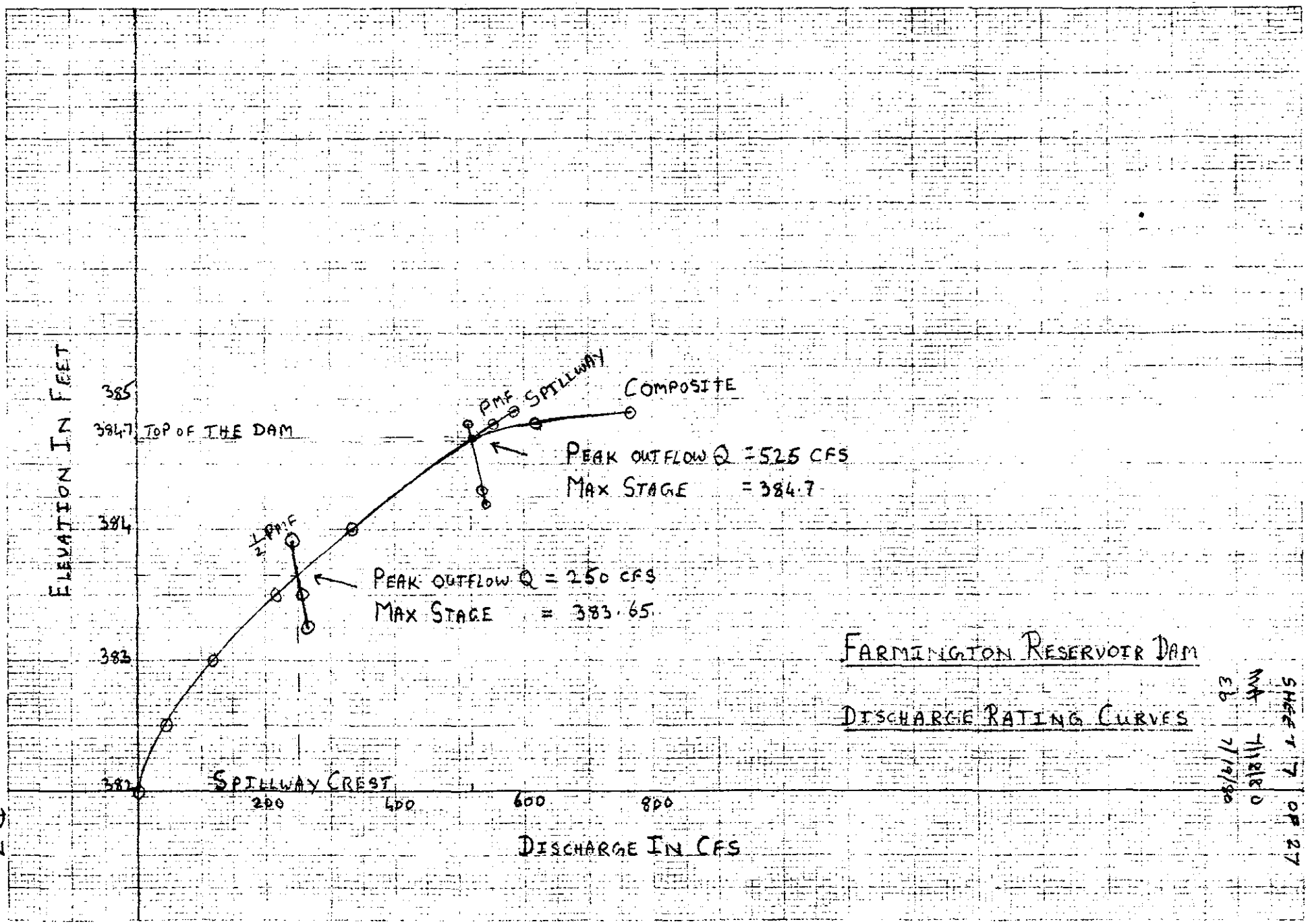
* USGS RECOMMENDED FORMULA FOR MORE PRECISE DISCHARGE OVER INCLINED DAM/EMBANKMENT CREST (REF: MEASUREMENT OF PEAK DISCHARGES AT DAM BY INDIRECT METHODS, USGS BOOK 3, CHAPTER A-5, PAGE 3-4, 1968)

TABULATION OF DISCHARGE RATES (CFS)

	ELVN.	SPILLWAY Q_1	DAM Q.	TOTAL Q
SPILLWAY →	382	0	0	0
	382.5	42	0	42
	383	118	0	118
	383.5	218	0	218
TEST FLOOD = ½ PMF →	383.65	250	0	250
	384	335	0	335
PMF @ TOD →	384.7	525	0	525
	384.8	555	64	619
	384.9	585	182	767

NOTE: FOR THE POOL ELEVATIONS ABOVE THE TOP OF DAM IN THE TABLE, Q_3 AND Q_4 FOR EMBANKMENTS ARE NEGLIGIBLE. ALSO, LOW-LEVEL OUTLET COULD NOT BE FOUND AND THEREFORE IS NOT INCLUDED IN THIS ANALYSIS.

DISCHARGE RATING CURVES FOR TOTAL Q (COMPOSITE) AND SPILLWAY ARE SHOWN ON SHEET 7.


ELEVATION IN FEET
385
384.7
384
383
382
TOP OF THE DAM
PMF SPILLWAY
COMPOSITE
PEAK OUTFLOW Q = 525 CFS
MAX STAGE = 384.7
1/2 PMF
PEAK OUTFLOW Q = 250 CFS
MAX STAGE = 383.65
SPILLWAY CREST
200
400
600
800
DISCHARGE IN CFS
FARMINGTON RESERVOIR DAM
DISCHARGE RATING CURVES

D-7

DETERMINATION OF PEAK OUTFLOW—
BY USING THE CORPS OF ENGINEERS "SURCHARGE STORAGE ROUTING" ALTERNATE METHOD:

FOR 315 CFS ($\frac{1}{2}$ PMF) THE DISCHARGE RATING CURVE GIVES ELVN = 383.92
FROM STAGE-STORAGE CURVE FOR THIS ELVN STORAGE = 30 AC. FT

$$STOR_i = \frac{30 \times 12}{0.26 \times 640} = 2.16'' \text{ RUN-OFF}$$

$$QP_i = QP_i \left(1 - \frac{STOR_i}{9.5}\right)$$

① STOR$_i$ INCHES	② $\left(1 - \frac{STOR_i}{9.5}\right)$	③ STOR$_i$ AC-FT ① $\times \frac{0.26 \times 640}{12}$	④ QP$_i$ CFS ② $\times$ 315	⑤ ELVN FROM STORAGE CURVE USING ③
1.5	0.84	20.8	265	383.25
1.75	0.82	24.3	258	383.5
2.16	0.77	30	242	383.92

COLUMNS ④ & ⑤ ARE PLOTTED ON DISCHARGE RATING CURVE AND

$\frac{1}{2}$ PMF PEAK OUTFLOW Q = 250 CFS
MAXIMUM STAGE = 383.65 NGVD
TOP OF DAM = 384.7 NGVD

∴ THE DAM IS NOT EXPECTED TO BE OVERTOPPED.

THE ROUTING IS ALSO DONE FOR INFLOW OF 625 CFS — BY USING THE CORPS OF ENGINEERS "SURCHARGE STORAGE ROUTING" ALTERNATE METHOD:

FOR 625 CFS (PMF) THE DISCHARGE RATING CURVE GIVES ELVN = 384.81

AND FROM STAGE-STORAGE CURVE FOR THIS ELVN, STORAGE = 44 AC-FT.

$$STOR_i = \frac{44 \times 12}{0.26 \times 640} = 3.17'' \text{ RUN OFF.}$$

$$QP_1 = QP_i \left(1 - \frac{STOR_i}{19}\right)$$

① STOR$_i$ INCHES	② $(1 - \frac{STOR_i}{19})$	③ STOR$_i$ AC-FT ① $\times \frac{0.26 \times 640}{12}$	④ QP$_i$ CFS ② $\times$ 625	⑤ ELV^N FROM STORAGE CURVE USING ③
2.50	0.87	34.7	544	384.2
2.75	0.86	38.1	538	384.3
3.00	0.84	41.6	525	384.7
3.17	0.83	44	519	384.81

COLUMNS ④ & ⑤ ARE PLOTTED ON DISCHARGE RATING CURVE AND

PEAK OUTFLOW Q = 525 CFS.

MAXIMUM STAGE = 384.7 NGVD

∴ THE DAM IS NOT EXPECTED TO OVERTOP.

BREACH ANALYSIS

DOWNSTREAM FAILURE HAZARD-

BREACH OUTFLOW $Q_b = \frac{8}{27} W_b \sqrt{g}\, y_o^{3/2}$ BASED ON CORPS OF ENGINEERS "RULE OF THUMB" GUIDANCE FOR ESTIMATING D/S DAM FAILURE HYDROGRAPHS.

W_b = 40% OF MID-HEIGHT LENGTH OF THE DAM.
MID-HEIGHT LENGTH OF THE DAM IS ESTIMATED = 600 FT.
BASED ON FIELD INFORMATION

$\therefore W_b = 0.4 \times 600$
$= 240$ FT.

ASSUMED WATER DEPTH AT TIME OF FAILURE y_o = 7.7 FT.
WITH POOL AT TOP OF DAM (EL. 384.7)

$Q_b = \frac{8}{27} \times 240 \times \sqrt{32.2} \times (7.7)^{3/2} = 8620$ CFS

PEAK FAILURE OUTFLOW $Q_P = Q_b$ + SPILLWAY DISCHARGE
$= 8620 + 525 = 9145$ CFS
SAY = 9200 CFS

ESTIMATED FAILURE FLOOD DEPTH IMMEDIATELY D/S FROM DAM
$= 0.44\, y_o$
$\cong 3.5$ FT

PERFORM D/S ROUTING OF PEAK FAILURE OUTFLOW

SELECT A SECTION AA 200' D/S OF THE DAM

USING MANNING'S EQUATION

$Q = \frac{1.486}{n} \times A \times R^{2/3} \times S^{1/2}$ $n = 0.06$ ASSUMED (FAIRLY FLAT)

$= 2.5 \times A \times R^{2/3}$ $S = 0.01$ ESTIMATED FROM USGS MAP

ELVN	A SQ.FT.	P	R = A/P	$R^{2/3}$	Q CFS
376	0	—	—	—	—
378	175	175	1	1	450
380	1000	650	1.54	1.33	3325
381	1481	696	2.13	1.65	6100
382	1995	735	2.71	1.94	9675

STAGE AREA AND STAGE DISCHARGE CURVES ARE PLOTTED FOR SECTION AA

FOR PEAK FAILURE OUTFLOW $Q_{P_1} = 9,200$ CFS, ELVN = 381.9 FROM STAGE DISCHARGE CURVE, AND STAGE AREA CURVE GIVES AREA = 1943 SQ.FT.

VOLUME OF REACH $V_1 = \frac{400 \times 1943}{43,560} \cong 17$ AC.FT.

THE SECTION AT AA IS ASSUMED TO BE UNIFORM BETWEEN THE DAM & RESERVOIR ROAD (400')

TRIAL $Q_{P_2} = Q_{P_1}\left(1 - \frac{V_1}{S}\right)$, WHERE S = TOTAL STORAGE TO TOP OF DAM = 64 AC.FT

$= 9,200\left(1 - \frac{17}{64}\right) \cong 6,750$ CFS.

FOR THIS Q_{P_2} THE STAGE DISCHARGE CURVE GIVES ELVN = 381.25

AND AREA = 1570 SQ.FT.

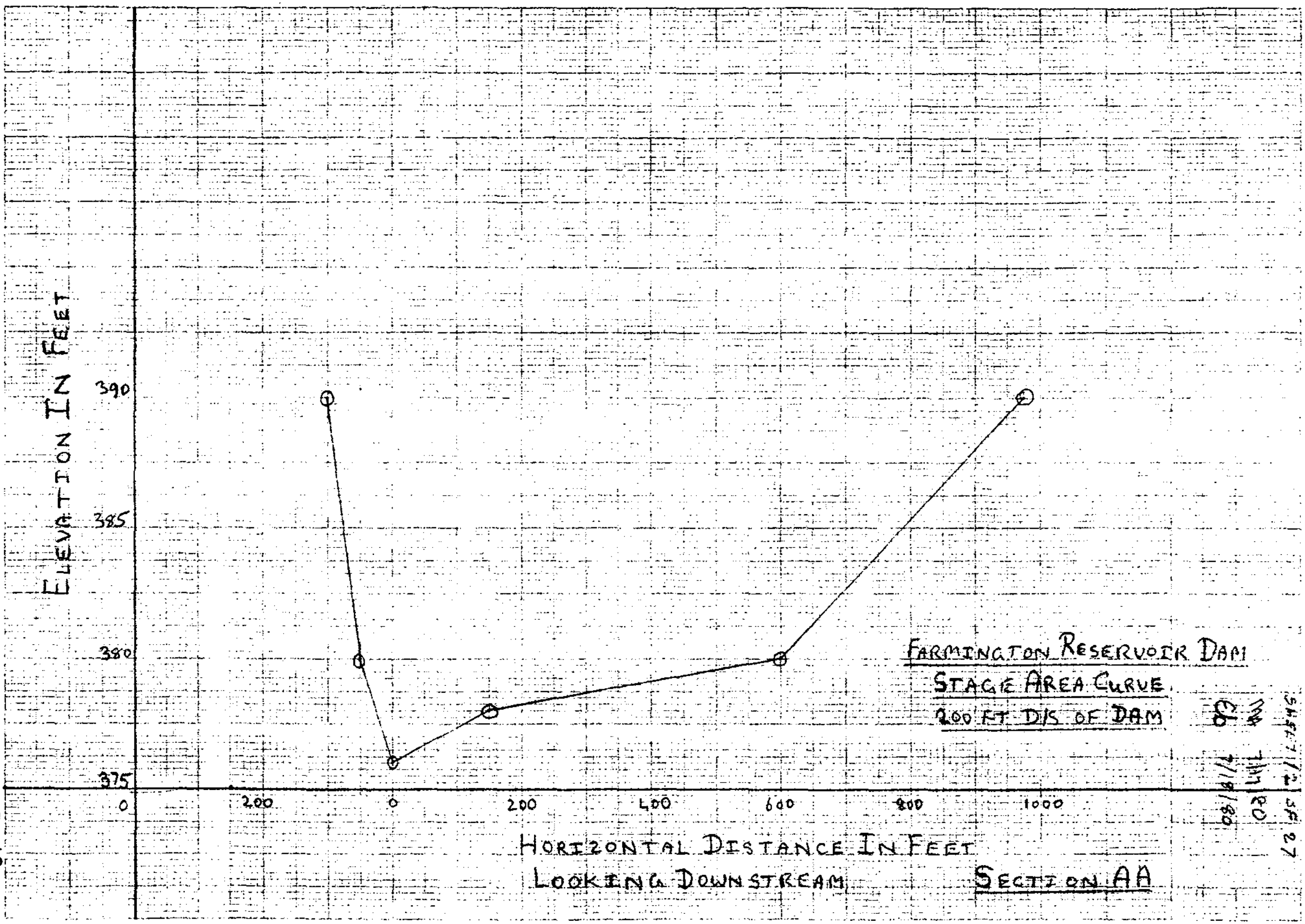
$V_2 = \frac{400 \times 1570}{43,560} \cong 14$ AC.FT.

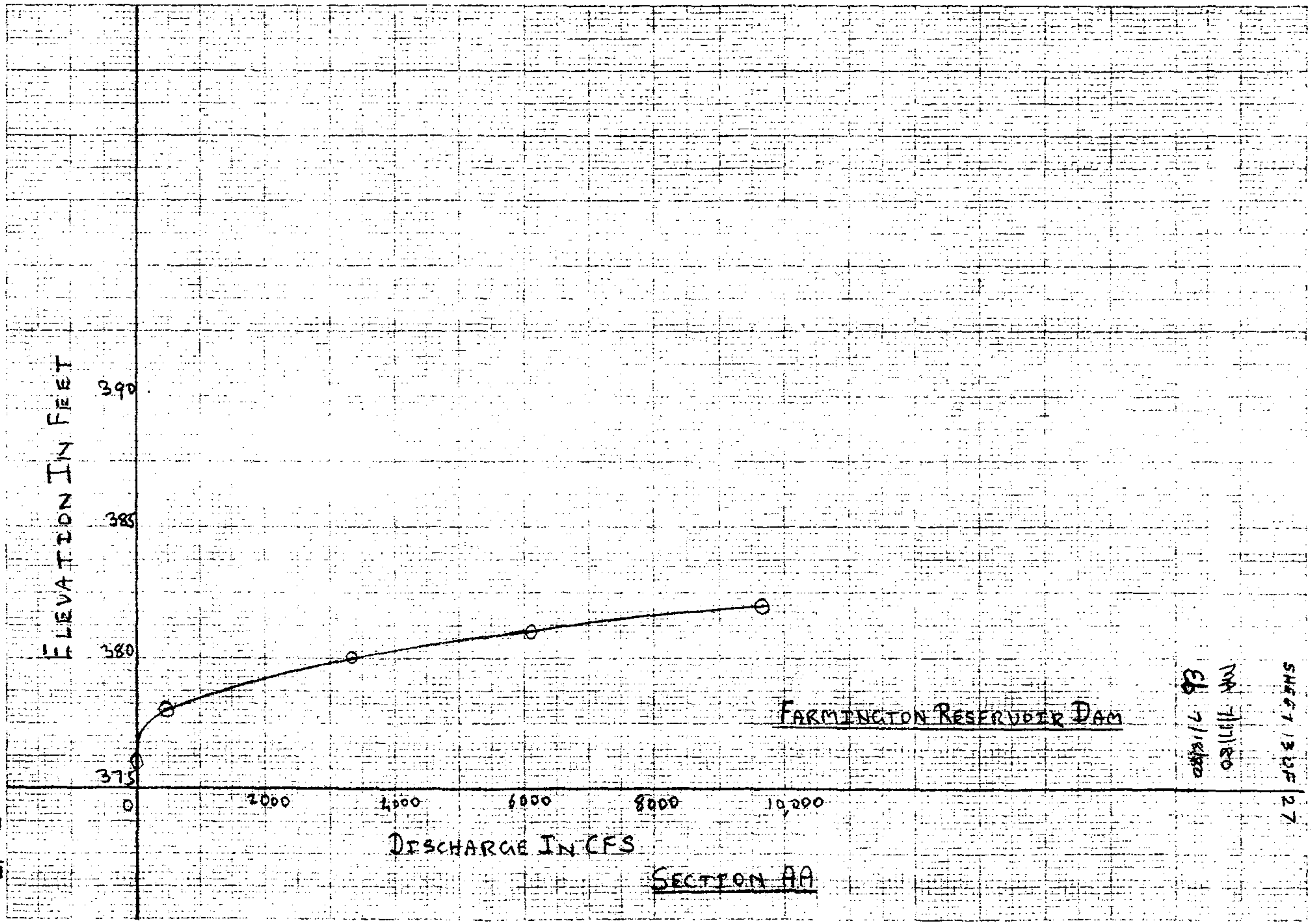
RECOMPUTING $Q_{P_2} = 9,200\left(1 - \frac{17+14}{2} \Big/ 64\right) \cong 7000$ CFS

AND FLOOD STAGE AT SECTION AA = 381.4

FLOOD DEPTH AT SECTION AA = 381.4 − 376 = 5.4 FT.

AND VELOCITY AT SECTION AA $= \frac{7000}{1675} = 4.2$ FPS


ELEVATION IN FEET
390
385
380
375
0
200
0
200
400
600
800
1000
HORIZONTAL DISTANCE IN FEET
LOOKING DOWNSTREAM
SECTION AA
FARMINGTON RESERVOIR DAM
STAGE AREA CURVE
200 FT D/S OF DAM



SHEET 13 OF 27

D-13

SELECTING A SECTION BB IMMEDIATELY D/S OF RESERVOIR ROAD

THIS COMPUTATION IS MADE TO ESTIMATE THE DEPTH OF FLOOD WATER IN THE VICINITY OF A HOUSE LOCATED ADJACENT TO THE STREAM.

USING MANNING'S EQUATION -

$$Q = \frac{1.486}{n} \times A \times R^{2/3} \times S^{1/2}$$

$n = 0.07$ ASSUMED
$S = 0.08$ ESTIMATED FROM USGS MAP

$A = 11.25\,H^2$

$P = 22.5\,H$

$R = \frac{A}{P} = \frac{11.25H^2}{22.5H} = 0.5\,H$

FOR TOTAL PEAK FAILURE OUTFLOW OF 7,000 CFS

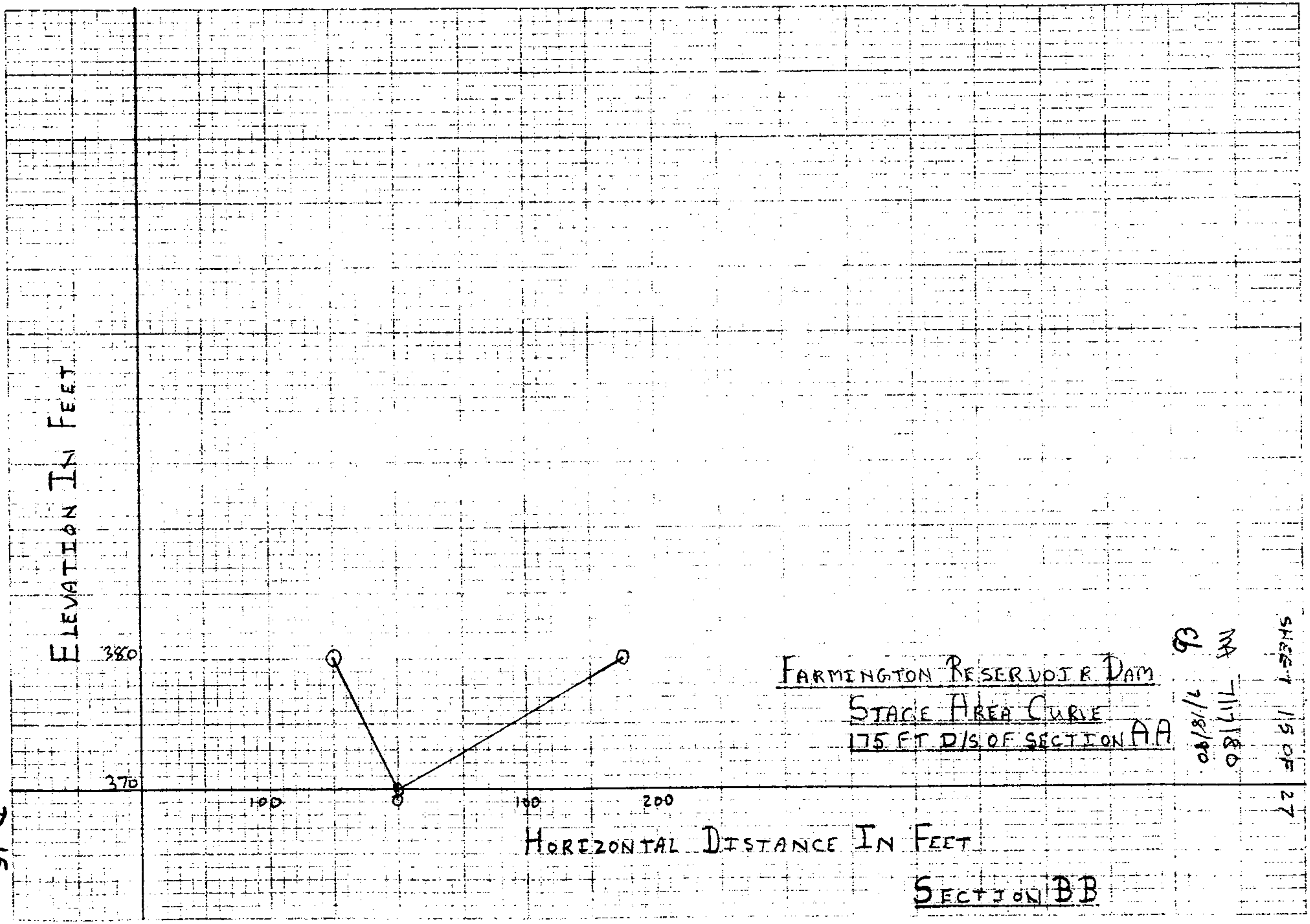
$$7{,}000 = \frac{1.486}{0.07} \times 11.25H^2 \times (0.5H)^{2/3} \times (0.08)^{1/2}$$

$\therefore H = \underline{6.8\ FT}$

FLOOD STAGE = 376.8 (CHANNEL BED = EL. 370)

AND VELOCITY $= \frac{7000}{527} = 13$ FPS

HENCE, THE HOUSE WOULD NOT BE IMPACTED SINCE IT IS LOCATED APPROXIMATELY 10+ FT ABOVE STREAM BED.


Elevation In Feet
380
370
100
100
200
Horizontal Distance In Feet
Farmington Reservoir Dam
Stage Area Curve
175 FT D/S of Section AA
Section BB
7/18/80
7/17/80
Sheet 15 of 27

SELECTING A SECTION DD 1450' DIS OF SECTION BB (500 FT DIS OF SECTION CC AND JUST ABOVE DORSET LANE) BETWEEN SECTIONS BB AND CC. THE STREAM IS NARROW WITH STEEP SLOPES (120' DROP IN LESS THAN 1000'); HENCE IT IS ASSUMED THAT THIS REACH WOULD NOT CONTRIBUTE TO ATTENUATION OF THE FLOOD VOLUME.

$Q = \frac{1.486}{n} \times A \times R^{2/3} \times S^{1/2}$ $n = 0.06$ ASSUMED

$= \frac{1.486}{0.06} \times A \times R^{2/3}$ $A = 0.067$ ESTIMATED FROM USGS MAP

EL.	A SQ FT	P	R = A/P	$R^{2/3}$	Q CFS
220	0	—	—	—	—
225	188	76	2.47	1.83	2,200
227	368	106	3.5	2.3	5,400
230	750	152	4.93	2.9	13,900

STAGE AREA AND STAGE DISCHARGE CURVES ARE PLOTTED FOR SECTION DD.
FOR PEAK FAILURE OUTFLOW QP_1 = 7,000 CFS. ELVN = 227.6
FROM STAGE DISCHARGE CURVE AND STAGE AREA CURVE
GIVES AREA = 460 SQ. FT.

VOLUME OF REACH $V_1 = \frac{500 \times 460}{43,560} \simeq 5$ AC. FT.

STORAGE REMAINING = $6.4 - \frac{17+14}{2} \simeq 4.9$ AC. FT.

TRIAL $QP_2 = QP_1\left(1 - \frac{V_1}{5}\right) = 7000\left(1 - \frac{5}{49}\right) = 6,300$ CFS

FOR THIS QP_2, ELVN = 227.5 AND AREA = 424 SQ. FT.

$V_2 = \frac{500 \times 424}{43,560} \simeq 5$ AC. FT.

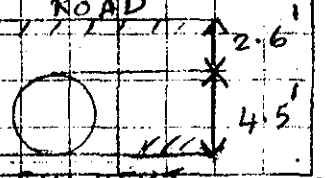
$QP_2 = 6,300$ CFS

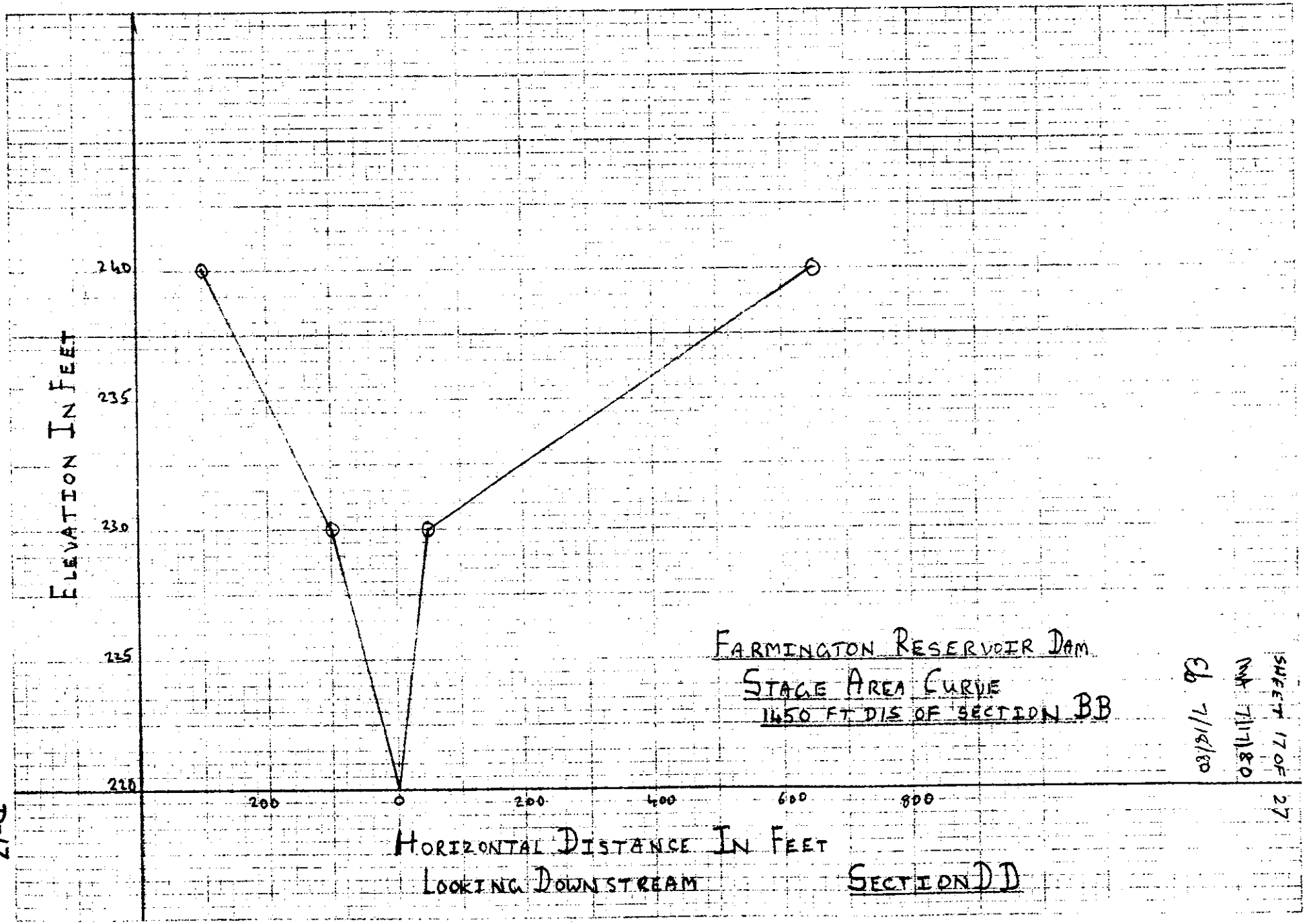
AND FLOOD STAGE AT SECTION DD = 227.5

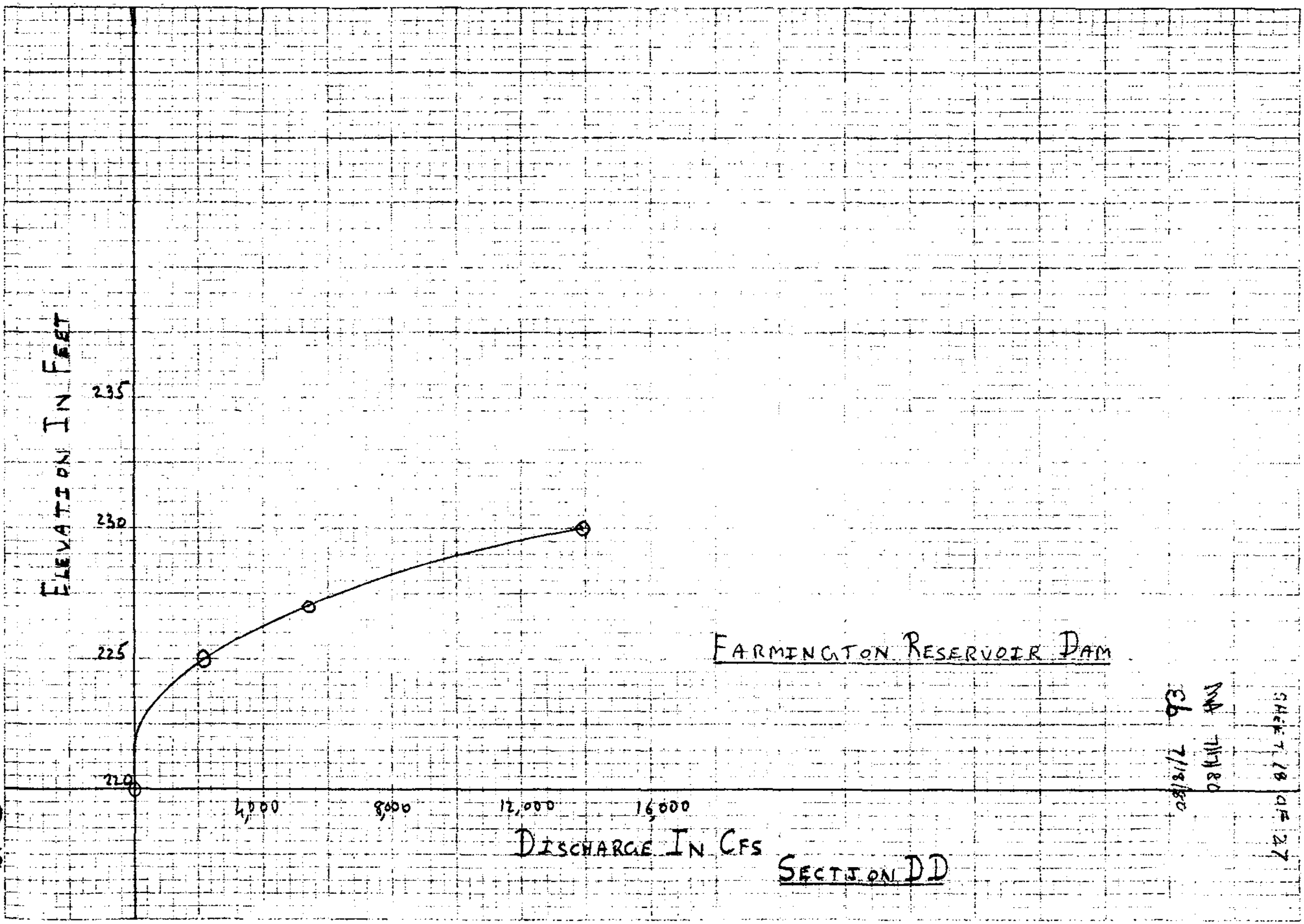
FLOOD DEPTH AT SECTION DD = 227.5 − 220 = 7.5 FT

AND VELOCITY AT SECTION DD = $\frac{6,300}{424} = 15$ FPS

THE DISCHARGE CAPACITY OF THE 4.5' DIA CULVERT = 170 CFS
FOR $\frac{HW}{D} = 1.6$ PER BUREAU OF PUBLIC ROADS. SCALE 2. MAY 1964.




ELEVATION IN FEET
240
235
230
225
220
200
0
200
400
600
800
HORIZONTAL DISTANCE IN FEET
LOOKING DOWNSTREAM
FARMINGTON RESERVOIR DAM
STAGE AREA CURVE
1450 FT DIS OF SECTION BB
SECTION DD
DWG 711180
SHEET 17 OF 27
03/19/80
GB



ELEVATION IN FEET
235
230
225
220
4,000
8,000
12,000
16,000
DISCHARGE IN CFS
FARMINGTON RESERVOIR DAM
SECTION DD
7/18/80
7/11/80
SHEET 18 OF 27

SELECTING A SECTION EE 300 FT D/S OF SECTION DD (BELOW DORSET LANE)

$Q = \frac{1.486}{n} \times A \times R^{2/3} \times S^{1/2}$ $n = 0.06$ ASSUMED

$= 4.75 \times A \times R^{2/3}$ $A = 0.033$ ESTIMATED FROM USGS MAP

EL.	A SQ. FT.	P	$R = A/P$	$R^{2/3}$	Q CFS
207	0	–	–	–	–
210	300	200	1.5	1.3	1750
211	517	237	2.21	1.7	3950
212	765	265	2.9	2	6900

STAGE AREA AND STAGE DISCHARGE CURVES ARE PLOTTED FOR SECTION EE.

FOR PEAK FAILURE OUTFLOW $Q P_1 = 6,300$ CFS. ELVN = 211.9 AND AREA = 739 SQ. FT.

VOLUME OF REACH $V_1 = \frac{300 \times 739}{43,560} \cong 5$ AC. FT.

STORAGE REMAINING $= 49 - \frac{5+5}{2} = 44$ AC. FT.

TRIAL $Q P_2 = Q P_1 \left(1 - \frac{V_1}{S}\right) = 6,300 \left(1 - \frac{5}{44}\right) \cong 5,600$ CFS

FOR THIS $Q P_2$, ELVN = 211.6 AND AREA = 662 SQ. FT.

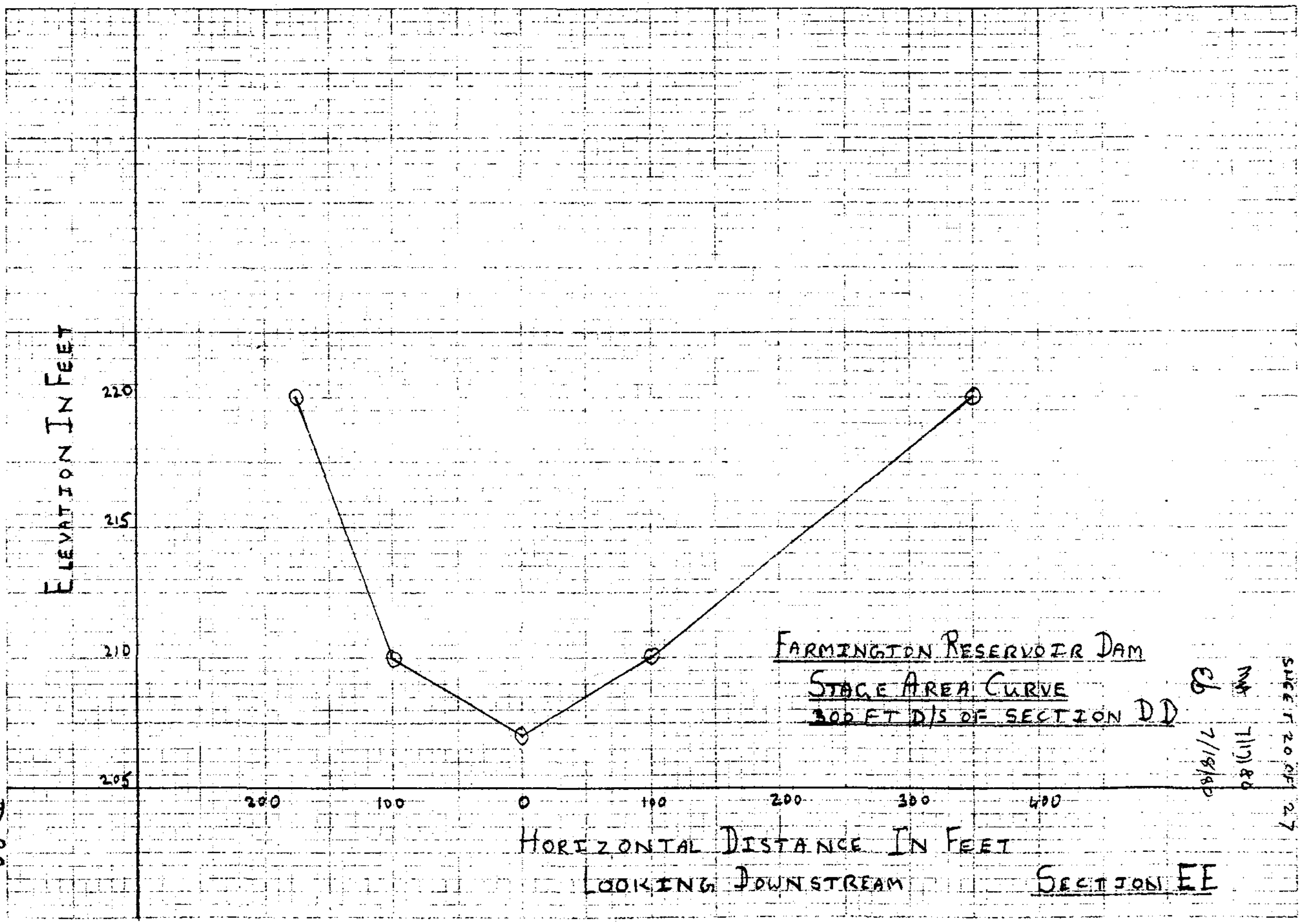
$V_2 = \frac{300 \times 662}{43,560} = 4.6$ AC. FT.

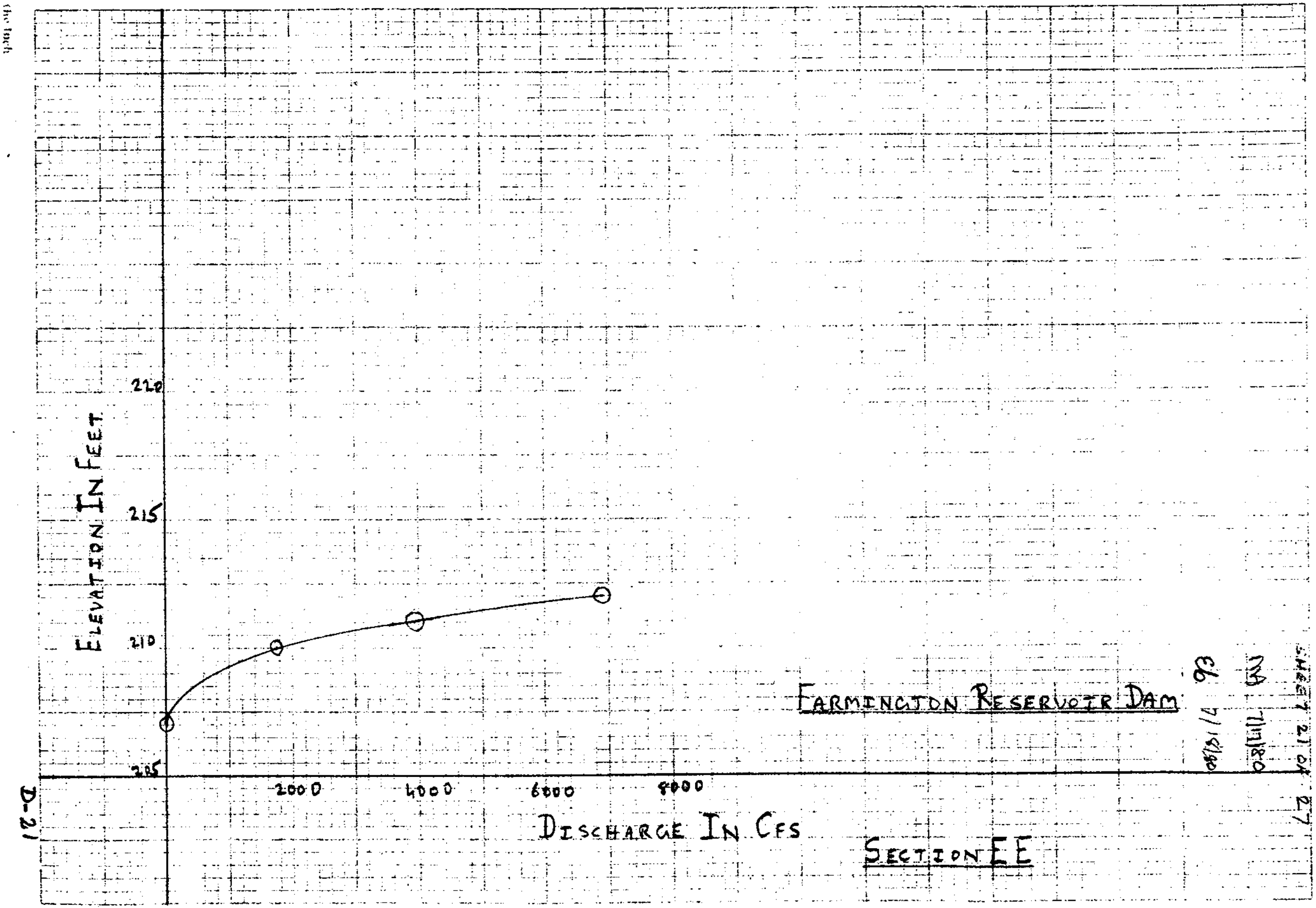
RECOMPUTING $Q P_2 = 6,300 \left(1 - \frac{\frac{5+4.6}{2}}{44}\right) \cong \underline{5,600 \text{ CFS}}$

AND FLOOD STAGE AT SECTION EE = 211.6

FLOOD DEPTH AT SECTION EE = 211.6 − 207 = 4.6 FT

AND VELOCITY AT SECTION EE $= \frac{5,600}{662} = \underline{8.5 \text{ FPS}}$


ELEVATION IN FEET
220
215
210
205
200
100
0
100
200
300
400
HORIZONTAL DISTANCE IN FEET
LOOKING DOWNSTREAM
FARMINGTON RESERVOIR DAM
STAGE AREA CURVE
300 FT D/S OF SECTION DD
SECTION EE
7/18/80
7/17/80
SHEET 20 OF 27


ELEVATION IN FEET
220
215
210
205
2000
4000
6000
8000
DISCHARGE IN CFS
FARMINGTON RESERVOIR DAM
SECTION EE
SHEET 21 OF 27
D-21

SELECTING A SECTION FF 250 FT D/S OF SECTION EE (JUST ABOVE THE POND)

$Q = \frac{1.486}{n} \times A \times R^{2/3} \times S^{1/2}$ $n = 0.06$ ASSUMED

$= 3 \times A \times R^{2/3}$ $S = 0.015$ ESTIMATED FROM USGS MAP.

EL	A, SQ. FT	P	R = A/P	$R^{2/3}$	Q CFS
198	0	—	—	—	—
200	250	250	1	1	750
201	532	315	1.7	1.42	2,300
203	1308	455	2.9	2	7,850

STAGE AREA AND STAGE DISCHARGE CURVES ARE PLOTTED FOR SECTION FF.

FOR PEAK FAILURE OUTFLOW $QP_1 = 5600$ CFS, ELVN = 202.5 AND AREA = 1088 SQ. FT.

VOLUME OF REACH $V_1 = \frac{250 \times 1088}{43,560} \cong 6$ AC. FT.

STORAGE REMAINING $= 44 - \frac{5+4.6}{2} \cong 39$ AC. FT.

TRIAL $QP_2 = QP_1 \left(1 - \frac{V_1}{S}\right) = 5,600 \left(1 - \frac{6}{39}\right) = 4,700$ CFS

FOR THIS QP_2, ELVN = 202.12 AND AREA = 940 SQ. FT.

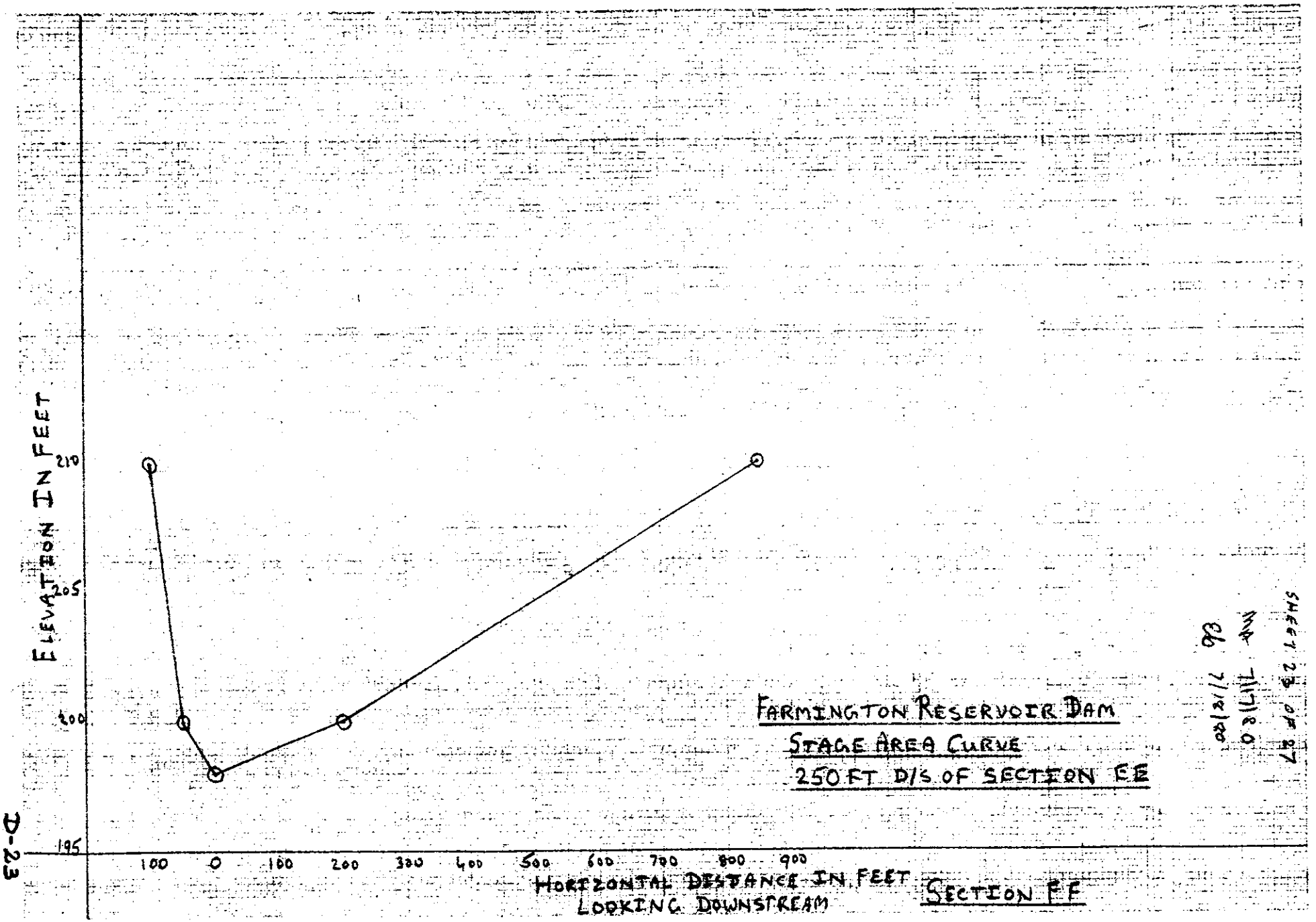
$\therefore V_2 = \frac{250 \times 940}{43,560} = 5.5$ AC. FT.

RECOMPUTING $QP_2 = 5,600 \left(1 - \frac{6+5.5}{39}\right) = 4,800$ CFS

AND FLOOD STAGE AT SECTION FF = 202.2 #

FLOOD DEPTH AT SECTION FF = 202.2 − 198 = 4.2 FT

AND VELOCITY AT SECTION FF $= \frac{4,800}{965} \sim 5$ FPS



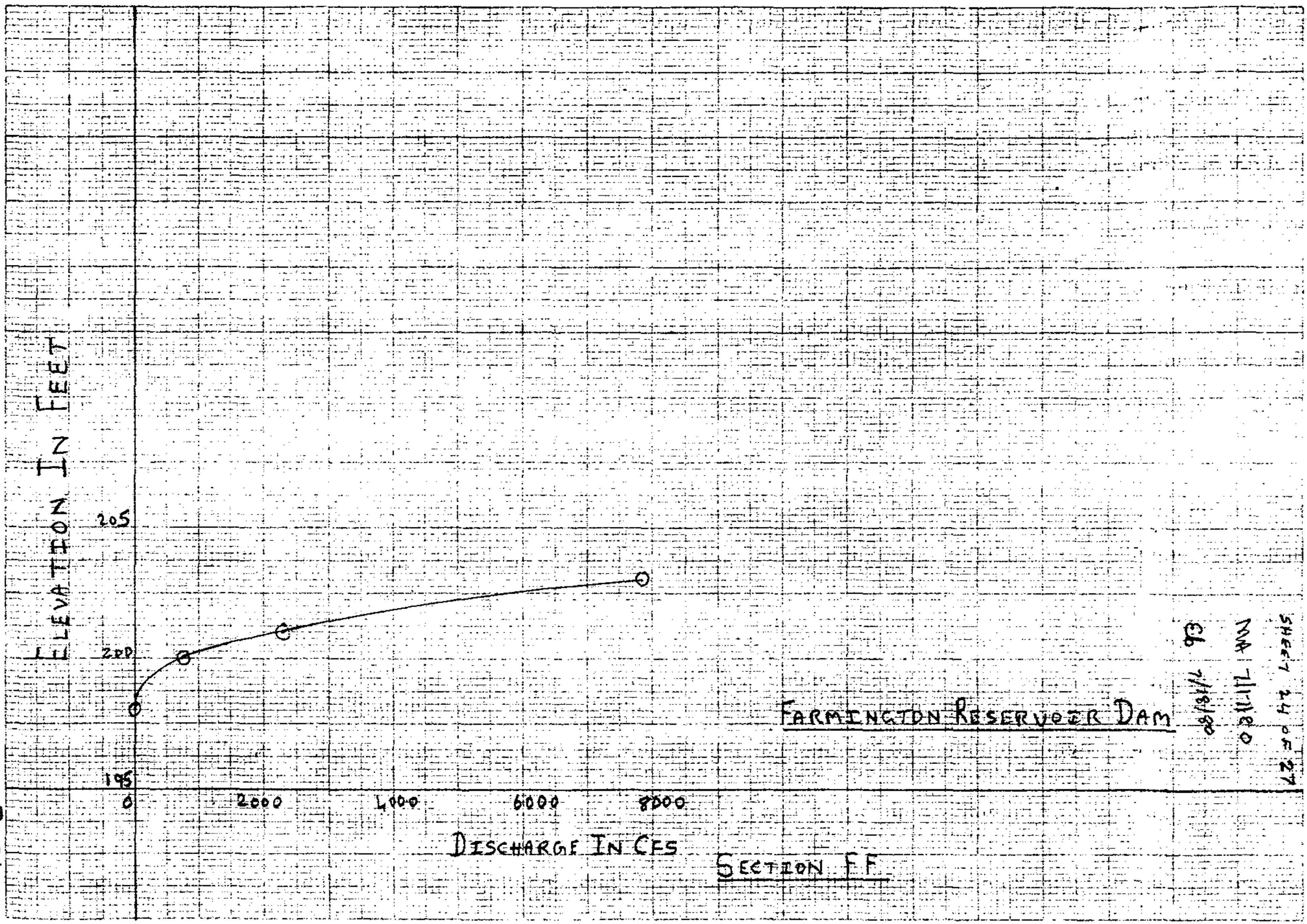
FARMINGTON RESERVOIR DAM
STAGE AREA CURVE
250 FT D/S OF SECTION EE

SECTION FF

Date 7/12/00

SHEET 23 OF 47

D-23


ELEVATION IN FEET
205
200
195
0
2000
4000
6000
8000
DISCHARGE IN CFS
FARMINGTON RESERVOIR DAM
SECTION FF
SHEET 24 OF 27
MA 7/11/80
EB 7/18/80

FAILURE HAZARD POTENTIAL

SUMMARY OF BREACH ANALYSIS RESULTS:

LOCATION	DISTANCE FROM DAM FT.	PEAK FLOW RATE CFS	FLOOD STAGE	FLOOD DEPTH FT.	VELOCITY FPS	STORAGE VOLUME REMAINING AC-FT
DAM	0	9200	380.5	3.5	—	64
AA	200	7000	381.4	5.4	4.2	49
BB	375	7000	376.8	6.8	13	49
DD	1825	6300	227.5	7.5	15	44
EE	2125	5600	211.6	4.6	8.5	39
FF	2375	4800	202.2	4.2	5	33

BETWEEN THE DAM AND RESERVOIR ROAD, SOME FAIRLY FLAT LAND EXISTS AND SECTION AA IS TAKEN THROUGH THIS AREA ADJACENT TO THE FILTER BEDS. THIS AREA ATTENUATES 15± AC-FT, WHICH IS EQUIVALENT TO 23% OF THE TOTAL BREACH VOLUME OF 64 AC-FT. THE RESERVOIR ROAD WOULD BE INUNDATED AND THE CULVERT ON THIS ROAD WOULD BE DAMAGED.

AT SECTION DD TAKEN ABOVE DORSET LANE, THE FLOOD DEPTH IS ESTIMATED TO BE 7.5' WITH A HIGH VELOCITY OF 15 FPS. AT LEAST ONE HOUSE, LOCATED ON THE RIGHT BANK OF THE STREAM WITH ITS' 1ST FLOOR ONLY 4± FT. ABOVE THE STREAM BED WOULD BE FLOODED WITH 3.5± FT. OF WATER. THE CULVERT ON DORSET LANE IS ESTIMATED TO HAVE A CAPACITY OF 1700 CFS WHEN COMPARED WITH 6300 CFS OF PEAK FLOW. THUS, THE CULVERT WOULD BE DAMAGED AND THE ROAD WOULD BE INUNDATED. THIS WOULD CUT OFF ACCESS TO SEVERAL HOUSES ON THE LOOP OF DORSET LANE. FURTHER,

ASSUMING THAT THE CULVERT STAYS, THE FLOOD WATER WOULD RISE, THUS IMPACTING OTHER HOUSES IN THE VICINITY.

FURTHER D/S AT SECTION FF, THE FLOOD STAGE IS ESTIMATED TO BE 202.2, FLOODING AT LEAST ONE HOUSE WITH 5± FT. OF WATER. THIS HOUSE IS 4± FT. ABOVE THE W.S. ELEVATION (EL. 193) OF THE SMALL POND LOCATED JUST D/S OF SECTION FF.

IN THE ENTIRE REACH, BETWEEN THE DAM AND SECTION FF, ONLY 45% OF DAM FAILURE FLOOD VOLUME IS EXPECTED TO BE ATTENUATED AND 33 AC.FT STILL REMAINS.

THUS, WITH AT LEAST TWO HOUSES FLOODED WITH 3.5 FT TO 5 FT OF WATER, TWO CULVERTS DAMAGED, TWO ROADS INUNDATED AND A POTENTIAL FOR FLOODING ADDITIONAL HOUSES ON DORSET LANE, WITH A POSSIBLE LOSS OF LIVES OF MORE THAN A FEW, A HIGH HAZARD POTENTIAL IS CONSIDERED LIKELY.

SUMMARY- HYDRAULIC/HYDROLOGIC COMPUTATIONS

TEST FLOOD PEAK INFLOW ½ PMF 315 CFS

(PARALLEL COMPUTATIONS HAVE BEEN MADE FOR PMF PEAK INFLOW AND RESULTS ARE SUMMARIZED BELOW)

PERFORMANCE AT PEAK FLOOD CONDITIONS:	PMF	½PMF
PEAK INFLOW CFS	625	315
PEAK OUTFLOW CFS	525	250
SPILL. CAP. TO TOP OF DAM CFS	525	525
SPILL. CAP. TO TOP OF DAM % OF PEAK OUTFLOW	100	210
SPILL. CAP. TO PEAK FLOOD ELVN. CFS	525	250
SPILL. CAP. TO PEAK FLOOD ELVN. % OF PEAK OUTFLOW	100	100
PERFORMANCE:		
MAXIMUM POOL ELVN NGVD	384.7	383.65
MAX. SURCHARGE HEIGHT ABOVE SPILL.CREST FT.	2.7	1.65
NON- OVERFLOW SECTION OF THE DAM OVERTOPPED	NO	NO

DOWNSTREAM FAILURE CONDITIONS:

PEAK FAILURE OUTFLOW CFS	9200
FLOOD DEPTH IMMEDIATELY D/S FROM DAM	3.5 FT
CONDITIONS AT INITIAL IMPACT AREA: SECTION DD (STREAM BED EL.220)	
ESTIMATED STAGE BEFORE FAILURE WITH 250 CFS	222.6 NGVD
ESTIMATED STAGE AFTER FAILURE WITH 6300 CFS	227.5 NGVD
ESTIMATED RAISE IN STAGE AFTER FAILURE ΔY_1	4.9 FT
CONDITIONS AT SECONDARY IMPACT AREA: SECTION FF (STREAM BED EL.198)	
ESTIMATED STAGE BEFORE FAILURE WITH 250 CFS	199.3 NGVD
ESTIMATED STAGE AFTER FAILURE WITH 4800 CFS	202.2 NGVD
ESTIMATED RAISE IN STAGE AFTER FAILURE ΔY_2	2.9 FT

PRELIMINARY GUIDANCE

FOR ESTIMATING

MAXIMUM PROBABLE DISCHARGES

IN

PHASE I DAM SAFETY

INVESTIGATIONS

New England Division
Corps of Engineers

March 1978

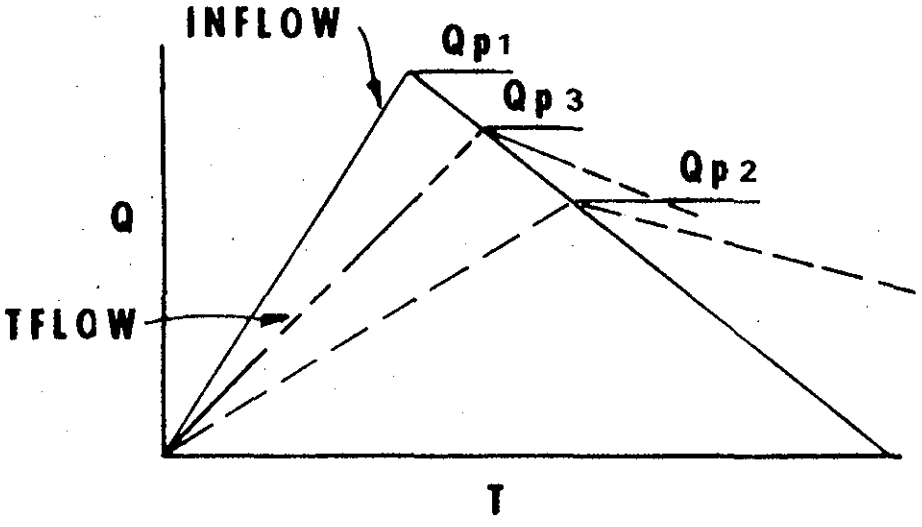
MAXIMUM PROBABLE FLOOD INFLOWS
NED RESERVOIRS

	Project	Q (cfs)	D.A. (sq. mi.)	MPF cfs/sq. mi.
1.	Hall Meadow Brook	26,600	17.2	1,546
2.	East Branch	15,500	9.25	1,675
3.	Thomaston	158,000	97.2	1,625
4.	Northfield Brook	9,000	5.7	1,580
5.	Black Rock	35,000	20.4	1,715
6.	Hancock Brook	20,700	12.0	1,725
7.	Hop Brook	26,400	16.4	1,610
8.	Tully	47,000	50.0	940
9.	Barre Falls	61,000	55.0	1,109
10.	Conant Brook	11,900	7.8	1,525
11.	Knightville	160,000	162.0	987
12.	Littleville	98,000	52.3	1,870
13.	Colebrook River	165,000	118.0	1,400
14.	Mad River	30,000	18.2	1,650
15.	Sucker Brook	6,500	3.43	1,895
16.	Union Village	110,000	126.0	873
17.	North Hartland	199,000	220.0	904
18.	North Springfield	157,000	158.0	994
19.	Ball Mountain	190,000	172.0	1,105
20.	Townshend	228,000	106.0(278 total)	820
21.	Surry Mountain	63,000	100.0	630
22.	Otter Brook	45,000	47.0	957
23.	Birch Hill	88,500	175.0	505
24.	East Brimfield	73,900	67.5	1,095
25.	Westville	38,400	99.5(32 net)	1,200
26.	West Thompson	85,000	173.5(74 net)	1,150
27.	Hodges Village	35,600	31.1	1,145
28.	Buffumville	36,500	26.5	1,377
29.	Mansfield Hollow	125,000	159.0	786
30.	West Hill	26,000	28.0	928
31.	Franklin Falls	210,000	1000.0	210
32.	Blackwater	66,500	128.0	520
33.	Hopkinton	135,000	426.0	316
34.	Everett	68,000	64.0	1,062
35.	MacDowell	36,300	44.0	825

MAXIMUM PROBABLE FLOWS
BASED ON TWICE THE
STANDARD PROJECT FLOOD
(Flat and Coastal Areas)

	River	SPF (cfs)	D.A. (sq. mi.)	MPF (cfs/sq. mi.)
1.	Pawtuxet River	19,000	200	190
2.	Mill River (R.I.)	8,500	34	500
3.	Peters River (R.I.)	3,200	13	490
4.	Kettle Brook	8,000	30	530
5.	Sudbury River.	11,700	86	270
6.	Indian Brook (Hopk.)	1,000	5.9	340
7.	Charles River.	6,000	184	65
8.	Blackstone River.	43,000	416	200
9.	Quinebaug River	55,000	331	330

ESTIMATING EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES



STEP 1: Determine Peak Inflow (Qp_1) from Guide Curves.

STEP 2: a. Determine Surcharge Height To Pass "Qp_1".

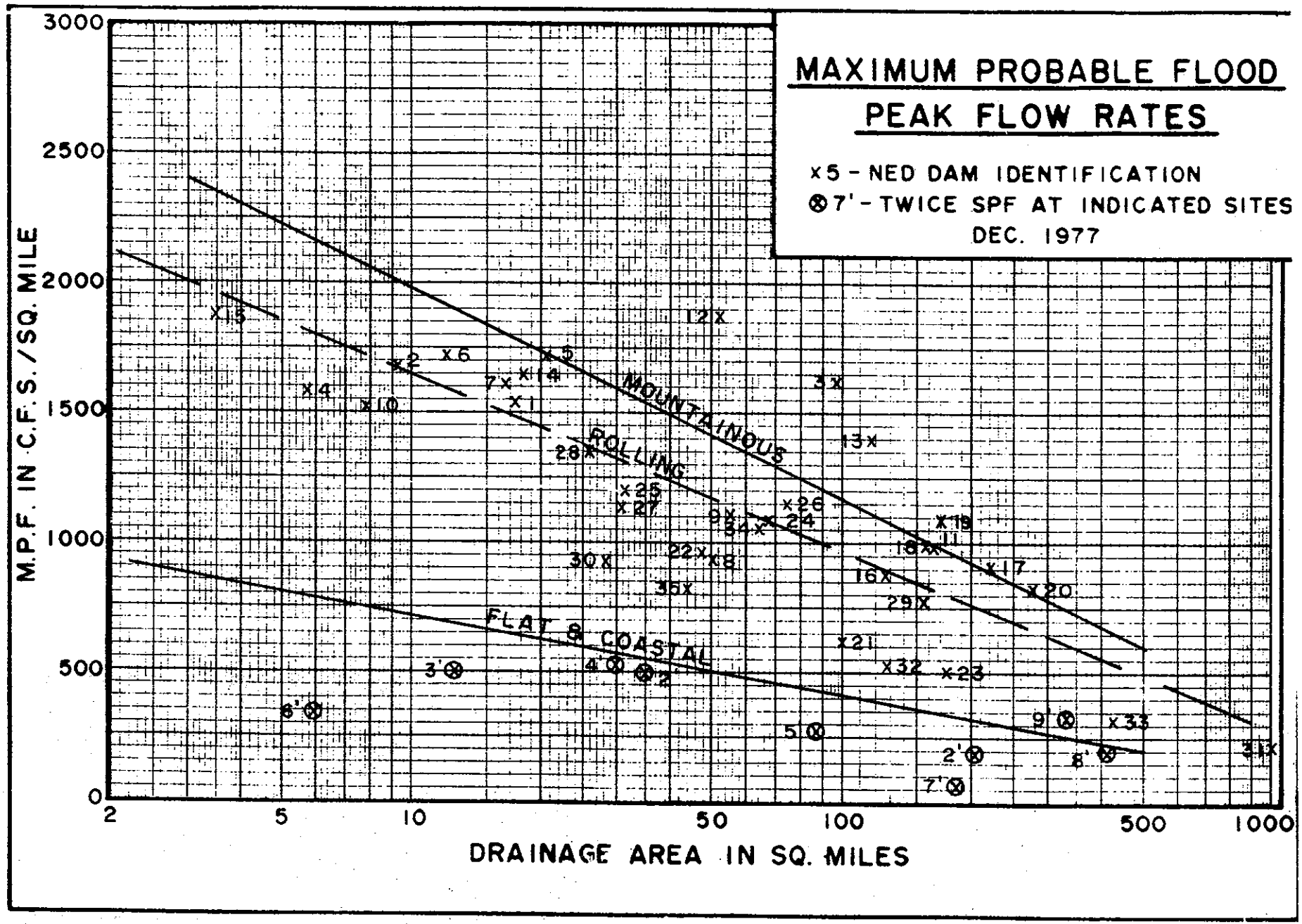
b. Determine Volume of Surcharge ($STOR_1$) In Inches of Runoff.

c. Maximum Probable Flood Runoff In New England equals Approx. 19", Therefore:

$$Qp_2 = Qp_1 \times \left(1 - \frac{STOR_1}{19}\right)$$

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Qp_2"

b. Average "$STOR_1$" and "$STOR_2$" and Determine Average Surcharge and Resulting Peak Outflow "Qp_3".


MAXIMUM PROBABLE FLOOD
PEAK FLOW RATES
x5 - NED DAM IDENTIFICATION
⊗7' - TWICE SPF AT INDICATED SITES
DEC. 1977
M.P.F. IN C.F.S./SQ. MILE
DRAINAGE AREA IN SQ. MILES
3000
2500
2000
1500
1000
500
0
2
5
10
50
100
500
1000
MOUNTAINOUS
ROLLING
FLAT & COASTAL
x15
x4
x10
x2
x6
x5
7x
x14
x1
12x
3x
13x
28x
x25
x27
x26
24
9x
34x
x18
11
18x
16x
29x
x17
x20
30x
22x
x8
35x
x21
x32
x23
x33
31x
3'⊗
4'⊗
2'⊗
6'⊗
5⊗
9'⊗
2'⊗
6'⊗
7'⊗

SURCHARGE STORAGE ROUTING SUPPLEMENT

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Qp_2"

b. Avg "$STOR_1$" and "$STOR_2$" and Compute "Qp_3".

c. If Surcharge Height for Qp_3 and "$STOR_{AVG}$" agree O.K. If Not:

STEP 4: a. Determine Surcharge Height and "$STOR_3$" To Pass "Qp_3"

b. Avg. "Old $STOR_{AVG}$" and "$STOR_3$" and Compute "Qp_4"

c. Surcharge Height for Qp_4 and "New $STOR_{Avg}$" should Agree closely

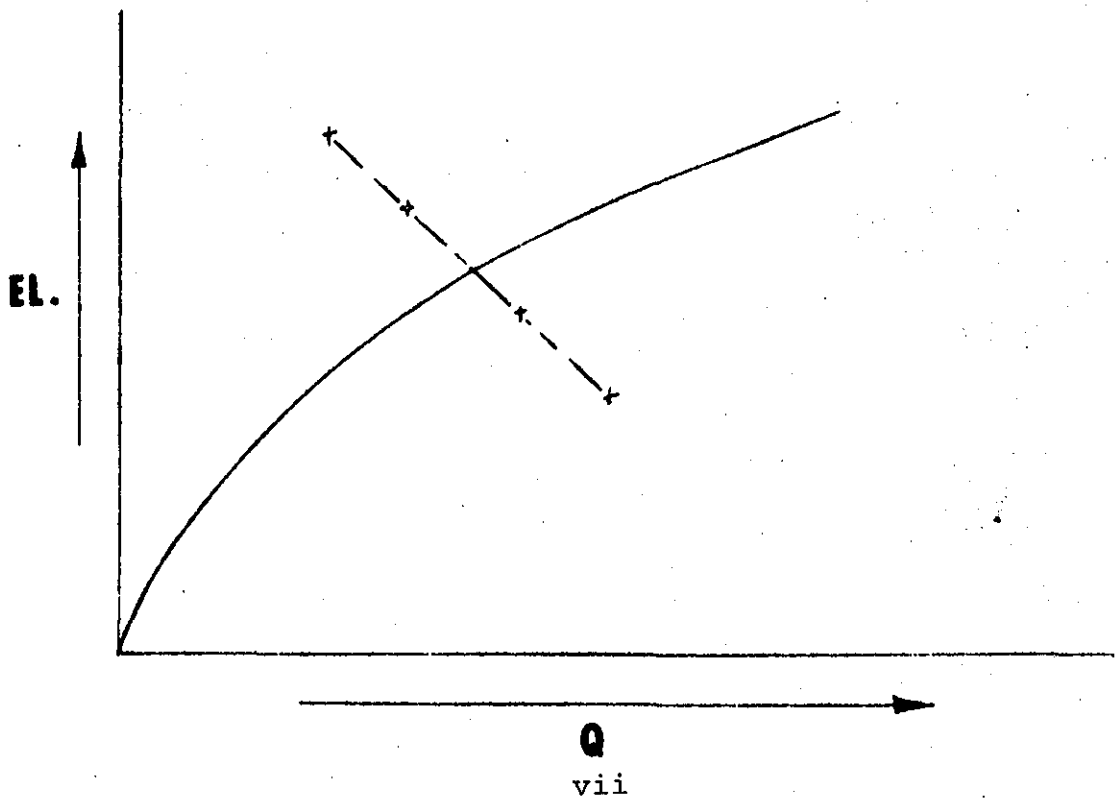
SURCHARGE STORAGE ROUTING ALTERNATE

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR}{19}\right)$$

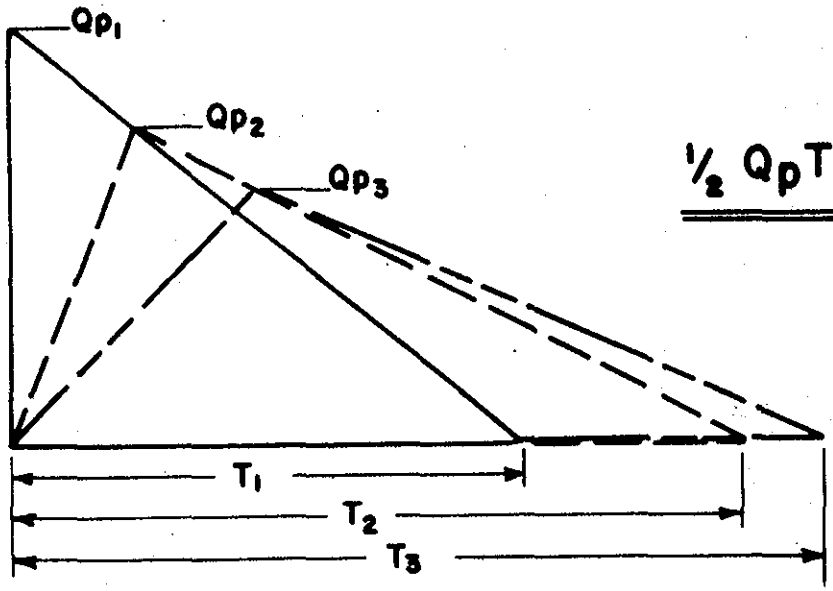
$$Q_{p2} = Q_{p1} - Q_{p1}\left(\frac{STOR}{19}\right)$$

FOR KNOWN Q_{p1} AND 19'' R.O.

Q_{p2}	STOR	EL.



"RULE OF THUMB" GUIDANCE FOR ESTIMATING DOWNSTREAM DAM FAILURE HYDROGRAPHS



$\frac{1}{2} Q_p T = 12 S$

STEP 1: DETERMINE OR ESTIMATE RESERVOIR STORAGE (S) IN AC-FT AT TIME OF FAILURE.

STEP 2: DETERMINE PEAK FAILURE OUTFLOW (Q_{p1}).

$$Qp_1 = \frac{8}{27} W_b \sqrt{g} Y_o^{3/2}$$

W_b = BREACH WIDTH - SUGGEST VALUE NOT GREATER THAN 40% OF DAM LENGTH ACROSS RIVER AT MID HEIGHT.

Y_o = TOTAL HEIGHT FROM RIVER BED TO POOL LEVEL AT FAILURE.

STEP 3: USING USGS TOPO OR OTHER DATA, DEVELOP REPRESENTATIVE STAGE-DISCHARGE RATING FOR SELECTED DOWNSTREAM RIVER REACH.

STEP 4: ESTIMATE REACH OUTFLOW (Q_{p2}) USING FOLLOWING ITERATION.

A. APPLY Q_{p1} TO STAGE RATING, DETERMINE STAGE AND ACCOPMANYING VOLUME (V_1) IN REACH IN AC-FT. (NOTE: IF V_1 EXCEEDS 1/2 OF S, SELECT SHORTER REACH.)

B. DETERMINE TRIAL Q_{p2}.

$$Qp_2(\text{TRIAL}) = Qp_1 \left(1 - \frac{V_1}{S}\right)$$

C. COMPUTE V_2 USING Q_{p2} (TRIAL).

D. AVERAGE V_1 AND V_2 AND COMPUTE Q_{p2}.

$$Qp_2 = Qp_1 \left(1 - \frac{V_{AVG}}{S}\right)$$

STEP 5: FOR SUCCEEDING REACHES REPEAT STEPS 3 AND 4.

APRIL 1978

APPENDIX E

INFORMATION AS CONTAINED IN THE NATIONAL INVENTORY OF DAMS

NOT AVAILABLE AT THIS TIME